PB82-105214

Evaluation of a Treatment Lagoon for Combined Sewer Overflow

Environmental Design and Planning, Inc.
Boston, MA

Prepared for

Municipal Environmental Research Lab.
Cincinnati, OH

Sep 81

U.S. Department of Commerce
National Technical Information Service
NTIS

EPA-600/2-81-196
September 1981

PB82-105214

EVALUATION OF A TREATMENT LAGOON FOR COMBINED SEWER OVERFLOW

by

Daniel J. Connick
William C. Pisano, Ph.D
Gerald L. Aronson

Environmental Design & Planning, Inc.
Allston (Boston), Massachusetts 02134

Grant No. 11023FAV

Project Officer

Richard P. Traver
Storm and Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory (Cincinnati)
Edison, New Jersey 08817

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA 22161

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

BEGIN FIRST LINE OF TEXT HERE
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD; BEGIN SECTIONS HERE
9 1/8"
BEGIN LAST LINE OF TEXT
OTTOM OF AGE AREA; UTSIDE MENSION OR TABLES ND ILLUS ATIONS
EPA Form 2350-6 (4-80)
(PREVIOUSLY EPA FORM 287)
U.S. GOVERNMENT PRINTING OFFICE
TYPING GUIDE SHEET

117

NOTICE

THIS DOCUMENT HAS BEEN REPRODUCED FROM THE BEST COPY FURNISHED US BY THE SPONSORING AGENCY. ALTHOUGH IT IS RECOGNIZED THAT CERTAIN PORTIONS ARE ILLEGIBLE, IT IS BEING RELEASED IN THE INTEREST OF MAKING AVAILABLE AS MUCH INFORMATION AS POSSIBLE.

TECHNICAL REPORT DATA

(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-600/2-81-196	2. ORD Report	3. RECIPIENT'S ACCESSION NO. PB82 105214
4. TITLE AND SUBTITLE EVALUATION OF A TREATMENT LAGOON FOR COMBINED SEWER OVERFLOW		5. REPORT DATE September 1981 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Daniel J. Connick, William C. Pisano, Gerald L. Aronson		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Environmental Design & Planning, Inc. 19 Fordham Road Allston (Boston), Massachusetts 02134		10. PROGRAM ELEMENT NO. 1BC822 11. CONTRACT/GRANT NO. 11023FAV
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory - Cin., OH Office of Research and Development U.S. Environmental Protection Agency Cincinnati, OHIO 45268		13. TYPE OF REPORT AND PERIOD COVERED Final July 1979 to October 198 14. SPONSORING AGENCY CODE EPA/600/14

15. SUPPLEMENTARY NOTES

Project Officer: Richard P. Traver FTS 340-6677 (201) 321-6677

16. ABSTRACT

This report summarizes the results of a two year study aimed at assesing the effectiveness of a 143 MG faculatative lagoon for treating combined sewer overflow and polishing secondary wastewater treatment plant effluent.

The lagoon performance was evaluated for changes in the degree of aeration. This parameter has the dual function of mixing the lagoon and increasing the oxygen levels above that provided by natural reaeration. The pollutant removal performance of the lagoon was evaluated for different discharge levels/detention periods. The effects of the stormwater loadings on lagoon performance were assesed so as to determine the buffering capacity of the lagoon for treating volumes of highly polluted intermittent flow.

A full scale 18 MGD application of state-of-the-art disinfection technology was utilized for bacterial reduction. Sequential applications of chlorine and chlorine dioxide were made to lagoon effluent waters for opimization of dose rates and delay times between applications.

Color illustrations are reproduced in black & white.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Combined sewerw, Overflows, Water pollution, Aerators, Aerobic processes, Anaerobic processes, Disinfection	Aeration, Faculatative lagoons, Chlorine, Chlorine dioxide	13B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21.
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE

DISCLAIMER

This report has been reviewed by the Municipal Research Laboratory, U.S. Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technologies and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment for public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research, a most vital communications link between the researcher and the user community.

The deleterious effects of storm sewer discharges and combined sewer overflows upon the nation's waterways have become of increasing concern in recent times. Efforts to alleviate the problem depend in part upon the development of integrated technologies involving non-structural best management practices with structural storage and treatment concepts.

This report presents the summary findings of a two year data collection effort aimed at assessing the pollutant removal effectiveness of the world's largest faculatative lagoon in East Chicago, Indiana, as a means for treating combined sewer overflows with a primary function of polishing secondary effluent from a municipal facility impacted by industrial waste. The lagoon has a surface area of 14.5 acres (5.9 ha) with an average water depth of 35 feet (10.6m) and aerated by 8 floating 50 hp aerators. Construction of the lagoon was completed in 1971. Recently the lagoon and the aeration equipment were revitalized and put into a state of readiness by the East Chicago Sanitary District for the post construction evaluation program.

Polishing of the dry weather secondary treatment plant effluent and treatment of combined sewer overflows were demonstrated during the evaluation program. In addition, optimization of aerator operations, and influent flow rates/mixing characteristics were performed. A full scale chlorine/chlorine dioxide disinfection system was implemented and evaluated for a two month period.

Francis T. Mayo
Director
Municipal Environmental
Research Laboratory

ABSTRACT

This report summarizes the results of a two year study aimed at assessing the effectiveness of the world's largest faculatative lagoon for treating combined sewer overflow and polishing secondary wastewater treatment plant (WWTP) effluent. The project was divided into five different activities.Each work phase evaluated a different aspect of the overall treatment scheme.

First, it was necessary to determine the physical characteristics of the lagoon as related to size, aerator locations and influent and effluent discharge levels. Physical dimensions of the lagoon and related structures were determined. Stage/discharge calibrations of influent and effluent channels were performed using velocity measurements and dye studies. Use of several pumps to lift supply of secondary treatment plant effluent to the lagoon made it necessary to calibrate each pump for the prevailing actual head loss conditions. This activity led to a detailed review of the lagoon influent characteristics. Analysis of the extremely large fluctuations of treatment plant flow led to the observation that the lagoon influent intermittently contained significant river water volumes. This problem is due to faulty pump operations at a bypass structure. Several recommendations to correct this problem and to optimize the efficiency of pumping operations and bypass control were made.

Second, the lagoon performance was evaluated for changes in the degree of aeration. This parameter has the dual function of mixing the lagoon and increasing the oxygen levels above that provided by natural reaeration. Evaluation of flow currents induced by a single aerator was performed, followed by dye tracer flow pattern determinations of the entire lagoon with several aeration schemes. Changes in inter-lagoon dissolved oxygen concentrations were evaluated for different configurations of operating aerators. Variability of oxygen levels at different locations and depths within the lagoon were determined at several flow and aerator combinations to ascertain the extent and location of anaerobic treatment. Treatment performance was evaluated at periods of aeration/no aeration.

Third, the pollutant removal performance of the lagoon was evaluated for different discharge levels/detention periods. WWTP effluent not treated in the lagoon was bypassed. Removal efficiencies were determined by daily sampling of the influent, efluent and at several locations within the lagoon. Solids,

organics, nutrients and several heavy metals were monitored. Lagoon treatment effectiveness was determined by averaging mass loadings over long periods. Variations in flow rates were considered by comparison of treatment realized at each flow rate relative to the overall removal considering bypassed flows.

Fourth, the effects of the stormwater loadings on lagoon performance were assesed so as to determine the buffering capacity of the lagoon for treating volumes of highly polluted intermittent flow. Immediate response of the lagoon on an effluent flow and concentration basis was made to determine initial washout effects. Removal efficiencies and effluent concentrations were also evaluated for changes from pre-storm levels.

Lastly, a full scale application of state-of-the-art disinfection technology was utilized for bacterial reduction. Sequential applications of chlorine and chlorine dioxide were made to lagoon effluent waters for optimization of dose rates and delay times between applications. Calibration of the chlorine feed and the chlorine dioxide production systems were performed to optimize the chemical usage. Bacteria counts were made before and after disinfection on a routine basis to determine the relationship in quantities of bacteria types and their susceptibility to disinfection. Specialized studies were performed to determine the optimum dosages of the two disinfectants and the effects of dose delay time.

11023FA ✓

This work was submitted in partial fulfillment of Grant No. 1120FAV by East Chicago Sanitary District, under the sponsorship of the U.S. Environmental Protection Agency. This report covers the period of July, 1979 to October, 1980 and work was completed October 1980.

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD; BEGIN SECTIONS HERE

CONTENTS

Disclaimer ii
Foreword iii
Abstract v
Figures ix
Tables xi
Abbreviations and Symbols xii
Acknowledgement xiv

1. Introduction 1
   1.1 Foreword 1
   1.2 Overview 1
   1.3 Synopsis of Work Completed 2

2. Conclusions 6
   A. General Overview 6
   B. Lagoon Operation 6
   C. Disinfection 8

3. Recommendations 9

4. Details of Lagoon and Related Appurtenances 11
   4.1 Foreword 11
   4.2 Lagoon/Aerators/Lift Stations Details 11
   4.3 Disinfection 18
      4.3.1 Review of Disinfection at ECSD Facility 18
      4.3.2 Discussion of Sequential Chlorine/Chlorine Dioxide Disinfection 19
      4.3.3 Design Basis/Details for Chlorine/Chlorine Dioxide System 20

5. Sampling Locations, Methods and Analytical Procedures . . . 23
   5.1 Introduction 23
   5.2 Details of Sampling Locations 23
   5.3 Sample Collection Methods 25
   5.4 Flow Gaging Equipment 25
   5.5 Lagoon Mixing Evaluation Methods 26
   5.6 Dissolved Oxygen Evaluation Methods 26
   5.7 Water Quality Parameter Coverage and Methods 27

6. Calibration of Lagoon Appurtenances 30
   6.1 Foreword 30
   6.2 Calibration of Lagoon Influent Pumps 30
   6.3 Influent Channel Stage/Discharge Relationships 32
   6.4 Effluent Stage/Discharge Relationships 34
   6.5 Analysis of Chlorination Chamber 40
   6.6 Exfiltration Rates 40
   6.7 Calibration of the Disinfection System 41

BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
3/8"

PAGE NUMBER
EPA-287 (Cin.)
(4-76)

CONTENTS (continued)

7. Analysis of Lagoon Mixing Characteristics 47
   7.1 Foreword 47
   7.2 Evaluation of Lagoon Mixing/Flow Characteristics with Various Aeration Schemes 47
   7.3 Evaluation of a Single Aerator Mixing Characteristics. 54
   7.4 Lagoon Dissolved Oxygen Concentration Patterns 56

8. Lagoon Treatment Rates 63
   8.1 Foreword 63
   8.2 Lagoon Performance Evaluation/Different Discharge Rates 63
   8.3 Comparative Analysis of Lagoon Removal Efficiencies With and Without Aeration 64
   8.4 Heavy Metal Removals 69
   8.5 Stormwater Impacts 72

9. Disinfection Program Results 83
   9.1 Foreword 83
   9.2 Routine Disinfection Evaluation Program 83
   9.3 Chlorine/Chlorine Dioxide Kill Studies 86
   9.4 Bench Scale Chlorine/Chlorine Dioxide Experiments . . 87

10. Porpourri of Miscellaneous Topics 92
   10.1 Foreword 92
   10.2 Rate of Lagoon Solids Deposition 92
   10.3 Solids Deposition in the CSO Influent Channel Clean Out Pit 92
   10.4 Algal Production and Inhibition 93
   10.5 Evaluation of BOD by Respirometer Technique 94
   10.6 Oxygen Uptake Rates 96

References 101

LIST OF FIGURES

Figure

1 Overview of Report 5

2 Schematic of East Chicago Lagoon and Related Appurtenances 12

3 Photographs of Lift Station and Lagoon Influent Channel 15

4 Photographs of Wet Weather Influent Channel and Aerators in Operation 16

5 Photographs of Lagoon Effluent Channel with DWF Slotted Discharge and Discharge to Little Calumet River with Bypass Pipes 17

6 Chlorine/Chlorine Dioxide Disinfection System Schematic 22

7 Location of Lagoon Sampling Stations 24

8 Stage Discharge Curves, Influent Channel 33

9 Stage Discharge Curves; Effluent Channel 36

10 Stage Discharge Curves at Effluent Weir and Detention Time/ Discharge Curves at Chlorination Chamber 39

11 Calibration Curves/Disinfection System 44

12 Chlorine Dioxide Production Curves 46

13 Preliminary Dye Tracer Study (3 mgd discharge) 48

14 Results of Dye Tracer Study (18 mgd discharge) 50

15 Lagoon Effluent Dye Tracer Concentrations versus Time 51

16 Isometric View of Lagoon Flow Patterns, 4 Aerators in Operation 53

17 Typical Aerator Induced Flow Pattern 55

18 Lagoon Effluent Concentrations, storm of 8/27/79 75

19 Sequential Chlorine/Chlorine Dioxide Disinfection Results (Full Scale) 88

20 Sequential Chlorine/Chlorine Dioxide Disinfection Results (Bench Scale Operation) 90

21 Typical Lagoon Influent/Effluent BOD Determinations 95

22 Lagoon Influent/Effluent Oxygen Uptake versus Time 97

LIST OF FIGURES (continued)

Figure

23 Oxygen Uptake at Station 4 versus Time 98

24 Benthic Oxygen Uptake Curves at Various Locations 100

LIST OF TABLES

Table

1 Summary of Monitoring Parameters/Frequency 28

2 Summary of Analytical Laboratory Methods 29

3 Summary of Average Pollutant Removal Effectiveness for Three Influent Flow Rates (3.0, 11.3 and 18.8 mgd) 65

4 Statistical Summaries of Pollutant Removal Effectiveness For Three Influent Flow Rates (3.0, 11.3 and 18.8 mgd) 66

5 Summary of Mean Pollutant Removal Rates Under Different Aeration Schemes 68

6 Statistics of Lagoon Heavy Metals Concentrations 70

7 Benthic Heavy Metal Results 71

8 Analyses of Lagoon CSO Treatment Effectiveness 77

9 Summary of Lagoon Wet Weather Treatment Performance 82

10 Statistical Analysis Lagoon Bacterial Levels 85

LIST OF ABBREVIATIONS AND SYMBOLS

cm - centimeter
Cm - cubic meter
cm/s - centimeter per second
cms - cubic meter per second
CSO - combined sewer overflow
dwf - dry weather flow
ECSD - East Chicago Sanitary District
fps - feet per second
ft - feet
ha - hectare
hp - horsepower
in - inch
kg - kilogram
m - meter
mg/l - milligram per liter
mgd - million gallons per day
MG - million gallons
% - percentage
sec - second
WWTP - Waste Water Treatment Plant

Symbols

BOD_e - Biochemical Oxygen Demand (respirometer)
BOD_5 - Biochemical Oxygen Demand (5 day)
Cd - Cadmium
Cl_2 - Chlorine
ClO_2 - Chlorine Dioxide
Cr - Chromium
Cu - Copper
Fe - Iron
Hg - Mercury
$NaCLO_2$ - Sodium Chorite
NO_3 - Nitrate
Pb - Lead

LIST OF ABBREVIATIONS (continued)

Symbols

O_2 - Oxygen
TP - Total Phosphorus
TS - Total solids
TSS - Total suspended solids
VSS - Volatile suspended solids
Zn - Zinc

ACKNOWLEDGEMENTS

The authors would like to commend the Environmental Design & Planning, Inc. (EDP) staff who worked so hard to make this program a success. An extreme debt of thanks is extended to Glen Gill, project EDP biologist, who faithfully collected and analyzed samples on-site for a full year period.

We would like to particularly express appreciation to the members of the EDP field crew, including Stephen Rock and Craig Anderson for their many hours of inclement weather endured.

Finally, last but certainly not least, we would like to express our sincerest appreciation to Rona Kurtz, Nora W. Scanlan and Nicky Kronenberg for their patience in typing this report.

East Chicago Sanitary District (ECSD) was the recipient of the grant from the U.S. Environmental Protection Agency. Dr. Timothy W. Raykovich and Dan Olson, Chief Chemist (ECSD) were the project managers for ECSD. Environmental Design & Planning, Inc., was the principle investigator to East Chicago for the project. Mr. Gerald Aronson, Executive Vice-President, EDP, was the project manager. Mr. Daniel Connick, EDP's supervisor of analytical services, supervised all the field work. Dr. William C. Pisano, President, EDP, and Daniel Connick prepared this report.

PROJECT OFFICER/SPONSOR

Richard P. Traver
Staff Engineer
Storm and Combined Sewer Section
Wastewater Research Division
Municipal Environmental
Research Laboratory
U.S. Environmental Protection Agency

Dr. Timothy W. Raykovitch
General Superintendent
East Chicago
Sanitary District
5200 Indianapolis Blvd.
East Chicago
Indiana 46321

CHAPTER 1

INTRODUCTION

1.1 Foreword

A general description and historical review of the overall demonstration project is presented in section 1.2. Synopsis of work completed in this study is given in section 1.3.

1.2 Overview

The East Chicago Sanitary District (ECSD) operates a wastewater treatment facility (WWTP) handling about 18 mgd of dry weather sewage from the City of East Chicago, Indiana. The waste is impacted by industrial wastes. Secondary treatment is provided consisting of conventional activated sludge. The plant has been historically impacted by industrial wastes and presently is being upgraded.

The East Chicago sewerage system is combined. In the past stormwater and bypassed sewage have impacted the Grand Calumet River with bacterial contamination. The combined sewer overflow contain both direct industrial heavy metal discharges as well as runoff from industrial areas.

In the late 1960's the ECSD applied for and received from the federal water program, a demonstration grant to construct and to evaluate the world's largest faculatative lagoon. The facility was intended to polish the secondary effluent from the WWTP and to provide preliminary treatment with disinfection to the major combined sewer overflow discharge from the City of East Chicago, Indiana. The project was to demonstrate the feasibility of a novel technique for stormwater pollution abatement together with supplementary treatment of municipal wastewaters in an industrialized urban area.

The City of East Chicago's objective was to meet the State, regional and federal quality requirements for the Grand Calumet River by improving quality from the East Chicago sewage treatment plant and from the Magoun Avenue pumping station for stormwater and by-passed sewage. Interception of the combined sewage overflow prevents discharge of raw sewage into the Grand Calumet River, thus eliminating the excessive bacterial pollution present after storms, and providing improvement in other respects. The improved quality of the Grand Calumet River Basin and of the lower reaches of Lake Michigan will enable increased industrial and recreational use of these waters. The value of the project to the federal water program was to demonstrate a proven wastewater treatment approach for handling combined sewer overflows.

The demonstration grant entailed detailed design, construction and one year documented evaluation program. The design and construction of the 700 foot by 900 foot basin with a maximum depth of 40 feet began in 1969 and was completed in 1972. The post construction evaluation program and final report work was not initiated. The project remained dormant until 1979. Further details are given in Chapter 4.

The ECSD and U.S. EPA desired to complete the original grant. Environmental Design & Planning, Inc. was engaged to perform the evaluation program and complete the final report. ECSD was to restore the facility into working order prior to initiating evaluation program, maintain and operate the facility together with all instrumentation during tthe evaluation program and assist EDP in sampling collection and analytical laboratory work.

The facility required significant alterations and modifications to ensure good working order. ECSD and its engineering consultant prepared and executed these changes. These alterations were finally completed near the end of the evaluation program due to inclement weather, equipment delivery delays and equipment malfunctions.

1.3 Synopsis of Work Completed

The work performed can be divided into three major phases. The first phase included a review of the physical characteristics of the lagoon. The second phase related to treatment and evaluation. The third phase included analysis of evaluation data and recommendations. The accomplishments of the three phases of work are as follows:

Phase One - Characteristics

- determined lagoon size, depth and aerator locations
- evaluated flow schemes to lagoon and bypass
- determined capacities of lift pumps
- developed flow calibration curves for the influent and effluent channels
- determined detention time/flow rate relationship of disinfection contact chamber
- determined exfiltration rate of lagoon
- calibrated disinfectant feed system and optimized chemical usage

Phase Two - Operational Evaluation

A. Treatment of secondary plant effluent

- 56 weeks of operational evaluation
- 13 pollution parameters analyzed every other day at 14 lagoon locations
- 7 heavy metals analyzed every other day at 4 lagoon locations
- 3 dye tracer studies to evaluate lagoon mixing
- 7 dissolved oxygen and temperature profiles within the lagoon
- 3 rates of lagoon flow evaluate
- 2 reoxygeration schemes evaluated

B. Treatment of CSO

- 4 storm period evaluations
- 7 pollutional parameters evaluated for short-term and long-term effects on lagoon performance

C. Disinfection

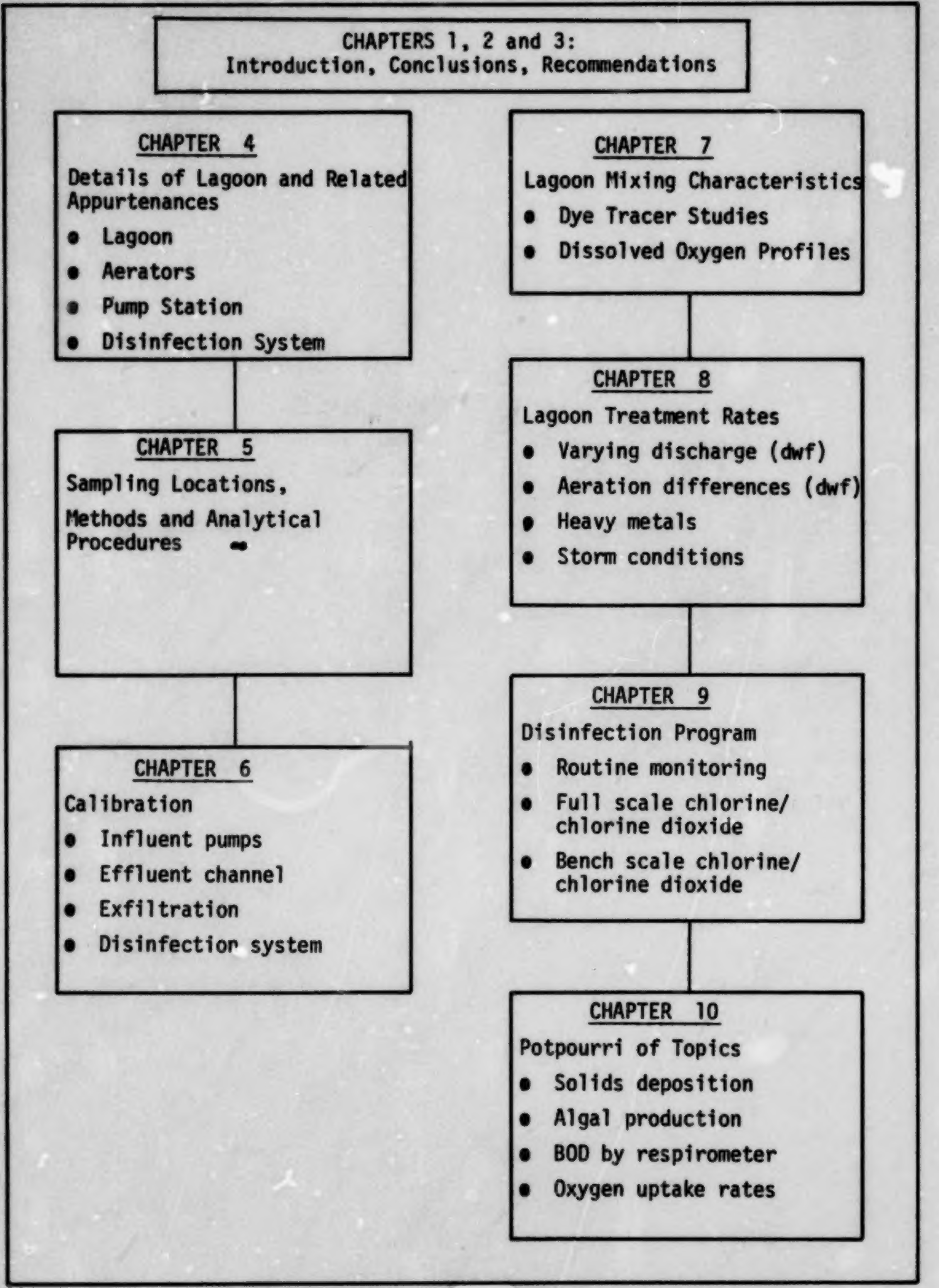
- 44 weeks of effluent bacteria monitoring
- 20 weeks of disinfection analysis
- 57 specialized disinfection studies to optimize dose ratio

Phase Three - Analysis

- all evaluation data processed and computer files prepared
- data evaluation to determine removal trends at various flow rates
- data evaluation to compare aeration effects on performance

- effects of storms evaluated
- disinfection dose requirements determined
- use of chlorine dioxide evaluated
- recommendations for lagoon facility modifications

The format of the report is depicted in Figure 1.



CHAPTERS 1, 2 and 3:
Introduction, Conclusions, Recommendations

CHAPTER 4

Details of Lagoon and Related Appurtenances

- Lagoon
- Aerators
- Pump Station
- Disinfection System

CHAPTER 5

Sampling Locations, Methods and Analytical Procedures

CHAPTER 6

Calibration

- Influent pumps
- Effluent channel
- Exfiltration
- Disinfection system

CHAPTER 7

Lagoon Mixing Characteristics

- Dye Tracer Studies
- Dissolved Oxygen Profiles

CHAPTER 8

Lagoon Treatment Rates

- Varying discharge (dwf)
- Aeration differences (dwf)
- Heavy metals
- Storm conditions

CHAPTER 9

Disinfection Program

- Routine monitoring
- Full scale chlorine/chlorine dioxide
- Bench scale chlorine/chlorine dioxide

CHAPTER 10

Potpourri of Topics

- Solids deposition
- Algal production
- BOD by respirometer
- Oxygen uptake rates

Figure 1. OVERVIEW OF REPORT

CHAPTER 2

CONCLUSIONS

Conclusions derived from this investigation are divided into three broad categories including, general overview; lagoon operation; and, disinfection.

A. General Overview

1. Results of the East Chicago Lagoon evaluation clearly indicate that relatively good treatment can be provided by a deep stabilization pond. During dry weather flow periods the lagoon provided treatment capabilities of 30-70% removal, depending on the parameter, for the entire secondary effluent.

2. Although removal efficiency decreased on a percentage basis with increased flow to the lagoon, the total mass of pollutants removed steadily increased. Therefore, the entire treatment plant effluent should be pumped to the lagoon.

3. Wet weather monitoring of the lagoon pollutant removal performance was limited. Average removals for three storm events approximated dry weather removal rates.

4. Influent, effluent and evaporation rates were monitored during extended dry periods and lagoon exfiltration was estimated to be negligible. Evidently organic debris had filled the voids in the land fill material used to form the basin, thus acting as a sealant.

5. Solids deposition rates were measured and estimated to be extremely low. Most of the solids from the ECSD WWTP are fine organic particles with low settling velocities.

B. Lagoon Operation

1. Data collection at high inflow rates was limited to the end of the evaluation period when water temperatures were relatively warm. Under this condition, optimum removal rates were produced by maximizing inflow rates. The effects of lower temperatures could not be directly determined at this flow rate due to the limited temperature variation during evaluation.

2. Evaluation of the mixing characteristics of a single aerator indicated that each unit is capable of influencing lagoon water flow patterns fo a distance in excess of one hundred feet. Aerators create flow patterns which extend to the lagoon bottom within close proximity of the units. Flow patterns were generally toward the aerators at depths below 10 feet and away from the aerators above this depth. Water velocity generally decreased with distance from the aerator and with depth.

3. Lagoon flow patterns are determined by the rate of inflow into the lagoon and by the number and locations of aerators in operation. Dye studies performed under several of these conditions demonstrated the general trend of induced-flow patterns. It was found that influent discharge from the 48-inch feed line tended to flow directly across the lagoon. The extent of the immediate flow across the lagoon was a function of initial influent momentum and thus flow rate and as expected,higher flows extended the flows further across the lagoon. It was found that without aeration, lagoon flow patterns allowed limited mixing of the interlagoon waters with most of the inflow rising to the surface and flowing to the effluent spillway with minimal detention time and limited treatment.

With the aerators in operation, the lagoon waters were adequately mixed with the influent. Operation of all eight aerators was found to completely mix the lagoon, producing high removal rates at optimum detention times. Analysis later indicated that only four aerators were required to produce this complete mix.

4. The other function of the aerators, reoxygenation of lagoon waters was found to be a necessity. Under most circumstances, four aerators were adequate but at times of high organic loading, the use of all eight appeared more advantageous. Continual aerator malfunction during the evaluation program limited the use of a preprogrammed routine for optimizing aerator operation.

5. The East Chicago Lagoon was designed as a deep basin to operate at an aerobic-anaerobic facility. Oxygen demand of the pollutional load was to have depleted 0_2 levels in the bottom waters whereas upper levels were to be mechanically reaerated. Under actual conditions, the lagoon was constructed 10 feet shallower than designed and the aerators imparted currents capable of mixing the lagoon bottom waters. As a result, oxygen levels remained nearly consistent from the surface to within two feet of the bottom. Current velocities near the bottom were recorded at over 0.3 fps (9cm/s) and indicated that with a deeper water body, an anaerobic zone may not be produced.

6. The aerators failed to produce the designed effect of reduced oxygen regeneration in the lower waters. The aerators are capable of adequately mixing surface lagoon waters to provide reasonable detention time. These aerators were more than adequate

capacity for the required functions of surface mixing and reaeration but the flow patterns they were designed to create, are not applicable to the envisioned operation of a deep water lagoon.

7. Floating surface aerators in the lagoon were subject to excessive icing with excessive winter malfunction problems.

C. __Disinfection__

1. Full scale (18 mgd) application of chlorine/chlorine dioxide disinfection produced comparable fecal coliform reduction levels as single chlorine application. However, no significant reduction in amount of chlorine required was achieved by using sequential C_2/ClO_2. The delay period between dosage was fixed at 45-90 seconds and could not be decreased.

2. Bench scale sequential Cl_2/ClO_2 was utilized to permit varying the delay period between disinfectant application. Decreased delay periods (15 - 30) seconds) did tend to decrease chlorine dosage rates for comparable fecal coliform reduction levels.

CHAPTER 3

RECOMMENDATIONS

1) The lagoon treats the entire effluent from the secondary plant.

2) Aerators should be replaced with units designed to limit subsurface turbulence or the present aerators should be equipped with bottom plates designed to limit updraw of bottom waters.

3) Sufficient aerators should be operated at all times to prevent short circuiting effects through the system.

4) Sufficient aerators should be operated at all times to meet reoxygenation needs.

5) The influent lift pumps should be automated to respond to changes in plant flow rate, as indicated by sump water levels.

6) Pumping of river water into the lagoon should be discontinued.

7) The sluice gate between the lift station and the river should be replaced with an adjustable overflow weir to prevent backflow of river water.

8) The influent channel should be redesigned to allow increased discharge of storm water flow. This reconstruction could be as simple as an above ground overflow at the location of the normal discharge point into the 48" pipe.

9) All stormwater should be pumped into the lagoon for treatment.

10) All aerators should operate during storm events.

11) Pumping from Magoun Ave. should be limited as much as possible by utilizing the inline storage capacity of the drainage system.

12) Large quantities of deposited solids in concrete channel between storm pump and influent channel should be cleaned after each storm.

13) Disinfection should include only chlorination until more reliable chlorine dioxide equipment is provided.

14) The chlorine dioxide system should be reconstructed to provide extensive mixing upon initial disinfection addition.

15) The operation of the $NaClO_2$ pump should be monitored regularly for diaphram wear, limiting pumping capacity.

16) Calibration of the $NaClO_2$ pump should be performed using the pH determination techniques developed in the study to eliminate syphoning and include backpressure effects and the need to dismantle system components.

17) The $NaClO_2$ pump should not be operated at the extreme ends of the stroke or speed ranges to prevent chemical plug flow through the chlorine dioxide generator.

18) ClO_2 should be monitored regularly to ensure optimum production by comparing theoretical maximum and actual production levels at the specific Cl_2 and $NaClO_2$ settings in use.

19) The flow proportioning system should be completed and calibrated and operated.

CHAPTER 4
DETAILS OF LAGOON AND RELATED APPURTENANCES

4.1 Foreword

Pertinent historical details and descriptions of the lagoon, aerators and pertinent lift stations are presented in Section 4.2. Prior chlorination practice and design details of a new chlorine/chlorine dioxide disinfection system are provided in Section 4.3.

4.2 Lagoon/Aerators/Lift Stations Details

Lagoon. Construction of the East Chicago treatment lagoon commenced in 1969 and was completed in 1971. During construction, serveral actions were undertaken that resulted in substantial changes from the original design. Important details directly affecting lagoon operations which required evaluation during this study are reviewed.

A schematic of the lagoon and related appurtenances is shown in Figure 2. The original lagoon design depicted a 700-foot by 900-foot basin (212m by 273m) with levied sides and a maximum water depth of 40 feet(12m). EDP field surveys indicated the post construction lagoon surface area remained unchanged, but the water depth was considerably less than originally planned. At the beginning of the evaluation period EDP determined a depth profile of the East Chicago lagoon. Originally this work was started using a supplied bottom sounding device. The unit was found to be not applicable since the the benthic layer muds created numerous sounding echoes. Depth measurements were then performed manually at 128 locations throughout the lagoon. Readings within 50 feet (15m) of the shore on any side reflected the 2 to 1 slope of the leveed sides. Depths throughout the remainder of the basin ranged between 25 to 32 feet with an overall average of 30.1 feet (9.1m). Volume of the post construction lagoon is estimated to be 142.2 MG (0.553 million cu.m.). Low solids removal during the evaluation phase negated the necessity for post evaluation depth sounding.

Materials used for levee construction were scrutinized as to their validity for this appliction. Although original plans required an impermeable lining, actual construction materials included assorted trash excavated from the site which had been a sanitary landfill. Although later soil borings indicated the "as is" material to be useless for this application, the method was approved during construction, based on the assumption that fine particles would eventually seal all holes during initial exfiltration periods. Analysis of exfiltration rates are reported in Chapter 6.

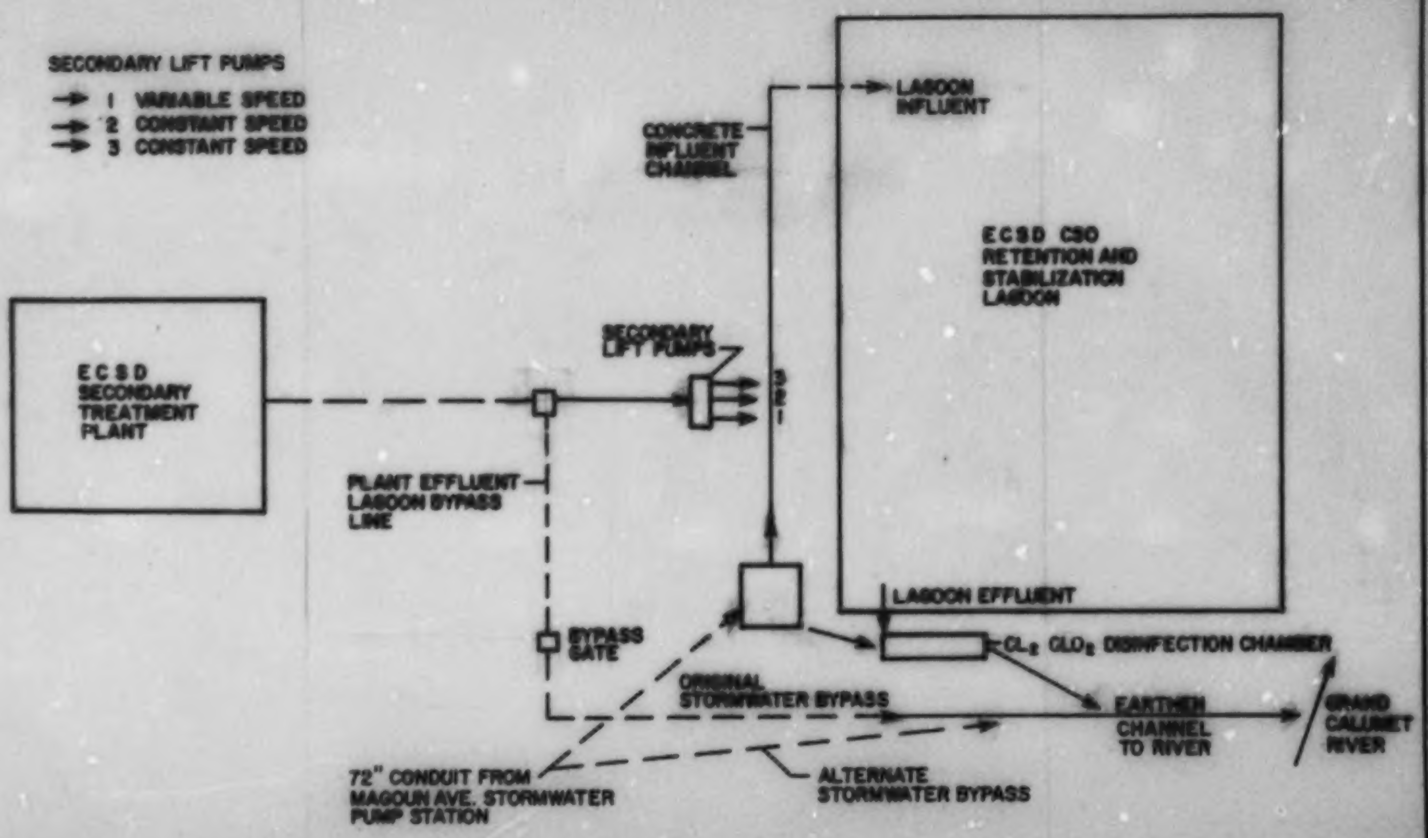

SECONDARY LIFT PUMPS
1 VARIABLE SPEED
2 CONSTANT SPEED
3 CONSTANT SPEED
E C S D SECONDARY TREATMENT PLANT
PLANT EFFLUENT LAGOON BYPASS LINE
BYPASS GATE
SECONDARY LIFT PUMPS
1
2
3
CONCRETE INFLUENT CHANNEL
LAGOON INFLUENT
E C S D CSO RETENTION AND STABILIZATION LAGOON
LAGOON EFFLUENT
CL_2 CLO_2 DISINFECTION CHAMBER
ORIGINAL STORMWATER BYPASS
EARTHEN CHANNEL TO RIVER
GRAND CALUMET RIVER
72" CONDUIT FROM MAGOUN AVE. STORMWATER PUMP STATION
ALTERNATE STORMWATER BYPASS

Figure 2. Schematic of East Chicago lagoon and related appurtenances.

Final construction of the influent system included a 9-foot wide by 8-foot high concrete channel along the north side with final discharge through a 48-inch (1.2m) conduit. It was originally intended that the influent be controlled via four outlets from the influent channel, with a combined capability of handling the entire Magoun Avenue Pumping Station flow, 208 cfs (7.8 cms). The capacity of the single 48-inch conduit is unknown, although calculations predict insufficient capacity to handle the CSO flow and overflowing of the influent spillway during major storms.

The lagoon effluent mechanism was to have incorporated four controlled flow outlets feeding a concrete channel within the south levee. A concrete apron located in the northwest corner with a central dry weather flow channel was substituted for the original system. At present, there are no means for directly controlling lagoon inflow and outflow. Influent flow enters the lagoon at a single point source and is directly related to pump rates. Effluent flow rates are directly related to lagoon water surface levels and discharges from a single point.

Aerators. The East Chicago lagoon was designed to treat local domestic and industrial dry weather wastes and combined sewer overflows during storm periods. The depth of the detention basin was designed to allow partial oxygen depletion yielding aerobic surface treatment with anaerobic bottom waters. The aerobic layer is provided and its depth controlled by surface aerators. Originally, nine 50 hp aerators were installed within the lagoon. However, due to multiple operational failures and the actual sinking of one unit, there are presently only eight aerators located in the lagoon.

The aerators are single speed 50 hp units electrically driven with waterproof cables connecting the units to onshore power transformers. Units are Model 450 Aqua Lator and incorporate a submerged impellar at a fixed distance below the surface. Each aerator unit is floated on a two piece fiberglass-covered polyethylene float assembly approximately one foot high (0.33m) and six feet (1.83m) in diameter. Aerators are approximately 18 inch (46 cm) in diameter and stand about 4 feet (1.2 m) high. Each unit is positioned by cable and rope assemblies interconnecting adjacent units and anchored at each embankment. No guards are provided to protect the upper section from induced spray nor is there any mechanism to manipulate the area of inflow to limit deep water draw.

Pump Stations. During rain periods, stormwater overflow from 20% of the City of East Chicago reaches the MagounAvenue Pumping Station. Prior to lagoon construction, stormwater overflow from this station was pumped directly to the Grand Calumet River. Since the lagoon was intended to handle all stormwater flow with no direct bypass to the river, the discharge main to the river was sealed off at completion of lagoon construction.

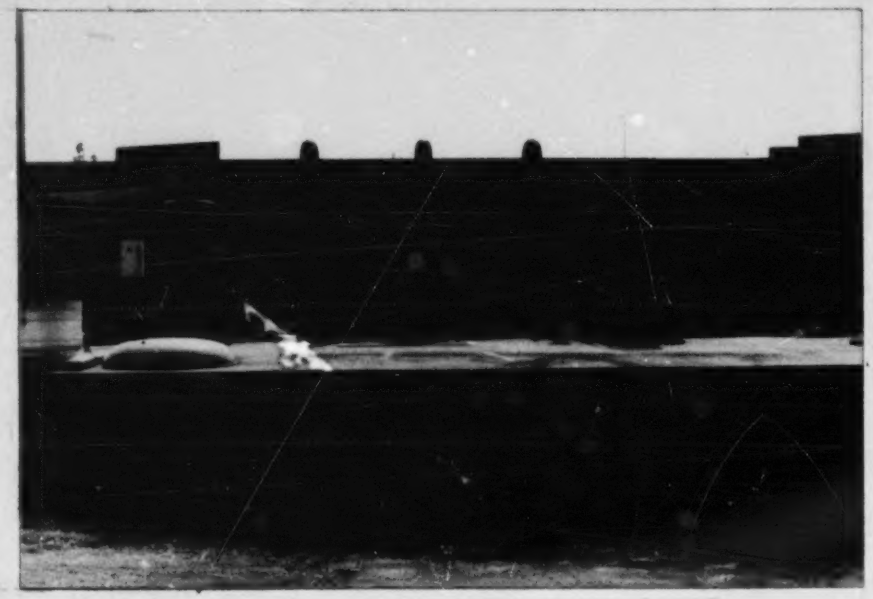
After completion of the lagoon in 1971, the first attempt to pump stormwater from the Magoun Avenue Station occurred in August, 1971. At that time, it was found that the five pumps at Magoun Avenue were of insufficient capacity to pump to the basin, since the increase in elevation created a total head of about 46 feet (14m). The pumps were previously adequate for the 26 foot (8m) lift to the Grand Calumet River. With the direct discharge line to the Grand Calumet River sealed off, stormwater could not be removed from the Magoun Avenue Station and flooding throughout the tributary sewerage system resulted. The initial solution was the installation of another direct discharge line to the Grand Calumet River. Stormwaters were therefore not pumped to the lagoon during the period from its completion in 1971 until 1977 when new pumps were installed.

A dry weather flow lift station was incorporated into the original facility design capable of pumping all the wastewater treated at East Chicago's secondary plant to the lagoon. Three fixed speed pumps were initially installed within the lift station. Two were capable of pumping 8 mgd (0.34 cms) each to the lagoon with the third unit having design capacity of 2 mgd (0.09 cms).

Multiple failures of the larger pumps occurred to the extent that the lagoon was inoperative. Rehabilitation of the lagoon and ancillary facilities required refurbishment of these pumps as well as those at the Magoun Avenue Station. Although the Magoun Avenue pumps were rebuilt and operating properly at the start of the evaluation project, the secondary lift station was not fully operable until well into the evaluation period.

Upgrading of the ECSD secondary WWTP effluent by stabilization in the lagoon was delayed until 1979. Prior to this time, a single submersible unit pumped plant effluent to the lagoon at a rate of about 3 mgd (0.13 cms). The balance of the WWTP effluent was bypassed and re-mixed with the lagoon effluent in an earthen ditch leading to the Grand Calumet River. The lift station containing the single pump was to be upgraded for the evaluation project to a capacity of 8 or 15 mgd, based on cost-effectiveness. The final decision was for the installation of two 8 mgd (0.34 cms) pumps such that the entire plant flow could normally be handled. Delivery and installation problems delayed operation of these pumps until May, 1979.

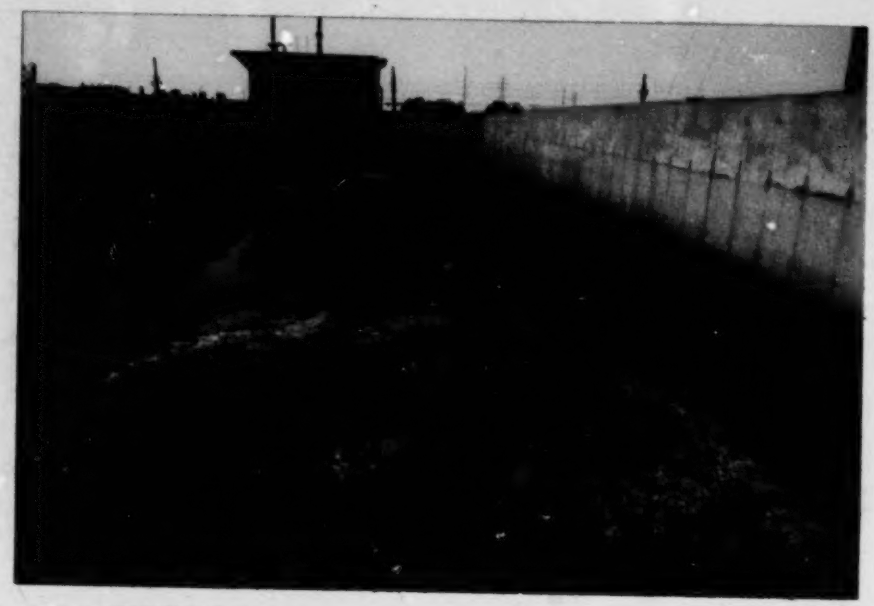
Photographs of various appurtenances are shown in Figures 3,4, and 5. The completed lift station for pumping the ECSD secondary effluent is shown in Figure 3-A. Three goose necks discharging into the influent channel are shown in the



A. Lift Station (Influent Channel in Background)

B. Influent Channel

Figure 3. Photographs of Lift Station and Lagoon Influent Channel



A. Wet weather channel from Magoun Avenue pump station (stilling basin in foreground, sluice to influent channel in background).

B. Aerators in operation

Figure 4. Photographs of wet weather influent channel and aerators in operation.

A. Lagoon effluent channel with DWF slotted discharge.

B. Discharge to Little Calumet River with bypass pipes.

Figure 5. Photographs of lagoon effluent channel with DWF slotted discharge and discharge to Little Calumet River with bypass pipes.

background of Figure 3-A. The influent channel is shown in Figure 3-B. The wet weather channel from the Magoun Avenue pump station is shown in Figure 4-A. Note the large accumulation of solids in the stilling basin in the foreground. The floating aerators in operation ar shown in Figure 4-B. The lagoon effluent channel with the DWF slotted discharge is shown in Figure 5-A. The lagoon effluent to the earthen ditch is shown in Figure 5-B. Bypass pipes to the lagoon are shown to the left of the discharge channel. Secondary plant effluent not pumped to the lagoon discharges through the bypass pipes and mixes with the lagoon effluent in the earthern ditch draining to the Grand Calumet River.

Other improvements supportive of the lagoon evaluation included ultrasonic flow meters and weirs and wave baffles for continuous recording of influent and effluent flow rates. The flow recording system was partially operational by September, 1979. Discharge calibration activities are described in Chapter 6.

4.3 Disinfection

Introduction

A review of chlorination at the ECSD facility and the economic justification for improved technique is presented in Section 4.3.1. A discussion of sequential application of chlorine/chlorine dioxide disinfection is given in Section 4.3.2. The basis for design and design details of a full scale sequential chlorine/chlorine dioxide unit for disinfecting lagoon effluent are presented in Section 4.3.3.

4.3.1 Review of Disinfection at ECDS Facility

The original disinfection design specified chlorine as the disinfecting agent and the construction of a chlorine contact chamber at the lagoon effluent. Actual construction alterations omitted the original chlorine contact chamber. Prior to refurbishment of the lagoon facilities, disinfection was conducted by chlorinating the lagoon effluent discharge (approximately 3 mgd) with sufficient dosage such that when the lagoon effluent was remixed with the bypassed discharge in the earthern ditch leading to the Grand Calumet River (see Figure 5-B), the bacterial standards at a point just downstream of mixing would be acceptable.

Up to 8000 lb/day (3636 Kg/day) chlorine were reported to be required for adequate disinfection of the secondary plant effluent. The application of chlorine at 8000 lb/day reflects dosages over 70 mg/1. Treatment plant effluent chlorine demands of 75 mg/1 were typically reported in 1977 and is based on the demand requirements of secondary effluent flows not receiving further treatment within the stabilization lagoon. The high cost of disinfection was a major factor in the consideration of sequential chlorine/chlorine dioxide treatment for disinfection. The combined effects of sequential dosing and reduced chlorine demand provided by lagoon treatment were expected to significantly lower disinfection costs.

During this project limited contact time was developed by installation of a steel weir in the effluent chamber. A high rate chlorine/chlorine dioxide system was installed to disinfect lagoon effluent waters but was not available for use until late in the evaluation study. The chlorine/chlorine dioxide system was operational using manual adjustment by May, 1979, but had not been completely set up for automatic flow proportioning by September, 1979.

During the final evaluation phase of this study chlorine application has dropped to 800 16/day (364 ks/day) for the lagoon effluent discharging at 18 mgd (0.8 cms). Chlorine residuals on the order of .5 mg/1 resulted with dosages calculated to be about 5 mg/1. A review of chlorination data indicated that from July to September, 1979, chlorine demand of the lagoon effluent averaged 2.4 mg/1. A chlorine demand test performed near the end of the evaluation period showed the lagoon influent demand to be 4.5 mg/1. The chlorine demand substantially decreased since 1977.

4.3.2. Dicussion of Sequential Chlorine/Chlorine Dioxide Disinfection

High chlorine demand levels of normal domestic wastewater require large quantities of chlorine to create a free Cl_2 residual. Free chlorine is consumed by several sources in aqueous solutions. Readily oxidizable substances in water create an immediate chlorine demand forming chloride ions. Chlorine concentrations remaining beyond the demand react with any ammonia present to form chlorimines

At the "break point" level, sufficient chlorine is added to just satisfy the chlorine demand of the water constituents with any further chlorine addition remaining in the free form. Free and combined chlorine yield varying rates of disinfection due to dissimilar bacteriacidal and viracidal properties. Although free chlorine is exceedingly more effective than the combined form, the expense of chlorination of wastewater to "break point" is unwarranted.

Disinfection with combined forms of chlorine at slower, less effective rates usually produces adequate reductions as statistically determined for wastewater discharges. The mechanisms of disinfection by oxidizing agents, such as chlorine, appears to occur through inactivation of enzymes by altering their chemical arrangement. The ease of disinfection seems to relate to the concentration of lipids in the cell envelopes or membranes.

Recently, investigation of disinfection techniques have increasingly emphasized the possible use of chlorine dioxide, which on a molar basis, is more effective than Cl_2 (1). As with Cl_2, ClO_2 disinfection efficiency appears to be directly related to lipid concentration . Chlorine dioxide reacts with fewer typical wastewater constituents than Cl_2, and those reactions that do occur are at slower rates. ClO_2 does decompose in aqueous solution, with the resulting ClO_2^- acting as a very weak disinfectant.

Disinfection analysis has shown comparable bacterial reductions to occur after 2 minutes contact time using dosages of 25 mg/1 Cl_2 or 12 mg/1 ClO_2.

Bench scale results also indicate sequential dosing may be a more effective disinfection technique (1). Multiple dosing with a single disinfectant demonstrated no improvement over the effects of a single equivalent dose. Sequential addition of different disinfectants was shown to enhance disinfection. It was hypothesized that after ClO_2 has been reduced to ClO_2^- any free Cl_2 present might oxidize the ClO_2^- back to ClO_2. As stated, ClO_2 is a much more effective disinfectant than ClO_2^- or Cl_2, and it is assumed that the reductions in ClO_2^- and Cl_2 concentrations will be offset by the degree of disinfection by renewed ClO_2.

The importance of delay time between sequential additions becomes significant when considering the reactions of free chlorine. Simultaneous reactions occur consuming free chlorine to renew ClO_2 and satisfy the aqueous solutions chlorine demand. The degree of ClO_2 renewal and thus disinfection enhancement, is directly related to the free Cl_2 concentration. The longer the delay period between dosage additions, the greater the decrease in free Cl_2, minimizing the overall regeneration effect.

4.3.3. Design Basis/Details for Chlorine/Chlorine Dioxide System

A disinfection system was installed in the lagoon effluent chamber with the design based on the conclusions of a recent bench-scale study (1). In that study high rate disinfection of combined sewer overflows with chlorine and chlorine dioxide was evaluated to aid in the design and operation of full-scale treatment facilities. The report concluded that a four logarithm reduction in indicator bacteria was obtained with dosages of 25 mg/1 of chlorine or 12 mg/1 of chlorine dioxide allowing two minute contact time. Disinfection was enhanced beyond the expected additive effect by using sequential addition of chlorine, followed by chlorine dioxide in 15 to 30 seconds.

Addition of 8 mg/l of chlorine followed by 2 mg/l of chlorine dioxide provided reductions similar to the single dosages noted above. The ECSD was designed on this basis.

A schematic of the chlorine/chlorine dioxide production and distribution system utilizing chlorine gas and on-site generation of chlorine dioxide in aqueous solution is presented in Figure 6. Chlorine is provided in banks of one-ton containers with an automatic switchover regulator to ensure uninterrupted chlorine gas supply.

The chlorination system design provides for conveying the gas under vacuum from the ton-containers to ejector assemblies. Conveyance under vacuum ensures the safety of the chlorination sytem as all chlorine gas flow ceases when the vacuum is interupted either purposely or accidentally.

Chlorine gas is used forboth the direct chlorination of lagoon effluent and for production of chlorine dioxide. As indicated in Figure 6 the chlorine gas separates after the automatic switchover regulator to feed the chlorination and the chlorine dioxide producton systems. Although flow to each system is indicated by a rotameter only total chlorine usage is permanently indicated by a flow recorder. A complete control panel regulates chlorine as flow to manually-set rates and is designed to provide automatic flow proportioning based on lagoon effluent flow rates.

Chlorine gas for direct chlorination passes directly to a remote shed near the effluent channel (see Figure 5-A) after being regulated at theconrol panel. Lagoon effluent used for the ejector water is pumped to the shed containing the gas ejector. This aqueous solution of chlorine is then pumped to the diffuser, located in the effluent chamber.

After flow regulation, chlorine gas supplied for chlorine dioxide production is mixed with city tap water and immediately passes to the ClO_2 generator. The chlorine dioxide precursor solution is a liquid sodium chlorite formulation containing 25 percent active $NaClO_2$ by weight.

This solution is supplied in bulk and stored in an outside, heated fiberglass tank. Sodium chlorite is pumped to the ClO_2 generator with a manually set chemical proportioning pump. Within the generator, the precursor solution reacts instantaneously with aqueous chlorine to produce ClO_2. This chlorine dioxide solution is currently fed to a diffuser at the end of the contact chamber providing the second step in the sequential disinfection system.

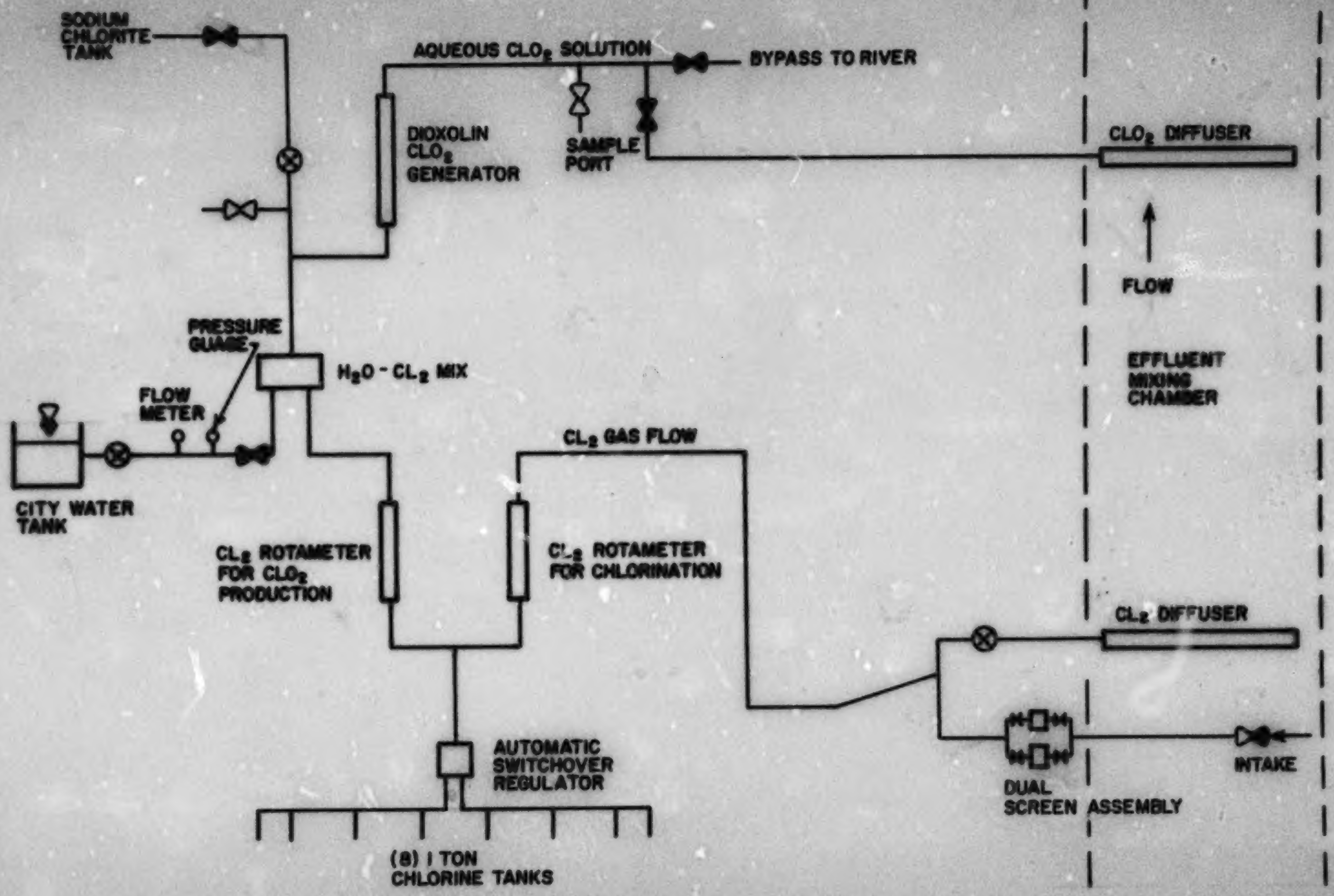

SODIUM CHLORITE TANK
AQUEOUS CLO_2 SOLUTION
BYPASS TO RIVER
DIOXOLIN CLO_2 GENERATOR
SAMPLE PORT
CLO_2 DIFFUSER
FLOW
EFFLUENT MIXING CHAMBER
PRESSURE GUAGE
H_2O - CL_2 MIX
FLOW METER
CITY WATER TANK
CL_2 GAS FLOW
CL_2 ROTAMETER FOR CLO_2 PRODUCTION
CL_2 ROTAMETER FOR CHLORINATION
CL_2 DIFFUSER
AUTOMATIC SWITCHOVER REGULATOR
DUAL SCREEN ASSEMBLY
INTAKE
(8) I TON CHLORINE TANKS

Figure 6. Chlorine / Chlorine dioxide disinfection system schematic.

CHAPTER 5

SAMPLING LOCATIONS, METHODS AND ANALYTICAL PROCEDURES

5.1 Introduction

This chapter describes sampling locations, field procedures, equipment and analytical methods. Details of the sampling locations are provided in section 5.2. Sample collection methods are given in section 5.3. Flow gaging calibration equipment is described in section 5.4. Methods for evaluating lagoon mixing and dissolved oxygen determinations are given in sections 5.5 and 5.6. Description of the monitoring program parameter coverage and analytical methods are given in section 5.7.

5.2 Details of Sampling Locations

The primary measurement program consisted of fifteen water sample locations monitored on a regular basis to identify the water characteristics at select locations in and around the lagoon. These locations are identified in Figure 7. Two stations were sampled hourly with automatic equipment providing daily composite samples. The stations at the influent and effluent points of the lagoon are designated as S1 and S2. Minimal diurnal flow variation at each of these points eliminated the need for flow proportioning. At all other locations, grab samples were collected. A sample station designated as S3 was located at the effluent side of the chlorine/chlorine dioxide disinfection contact chamber. All other samples were to be collected from within the lagoon. Four stations were located at the intersection points of two (east-west) lines running parallel to the long side of the lagoon at one-fourth the width from each side, and two (north - south) lines running perpendicular to the above lines at one-third the length from each end. Samples were collected at 10 feet below the surface, at 25 feet below the surface and at one foot from the bottom. These stations were designated as the "E", "H" and "B" series, respectively.

Specialized dye tracer studies utilized another sampling scheme in which thirty-two sampling sites were located at the intersection of four equally spaced lines running parallel to the long side and eight equally spaced lines run perpendicular to these.

At station S1, the automatic influent sampling device was located at the location of the flow sensing equipment at a point about one-half way down the concrete influent channel. Effluent samples prior to chlorination were taken from within the dry weather spillway in the upper reaches where flow velocities were minimal.

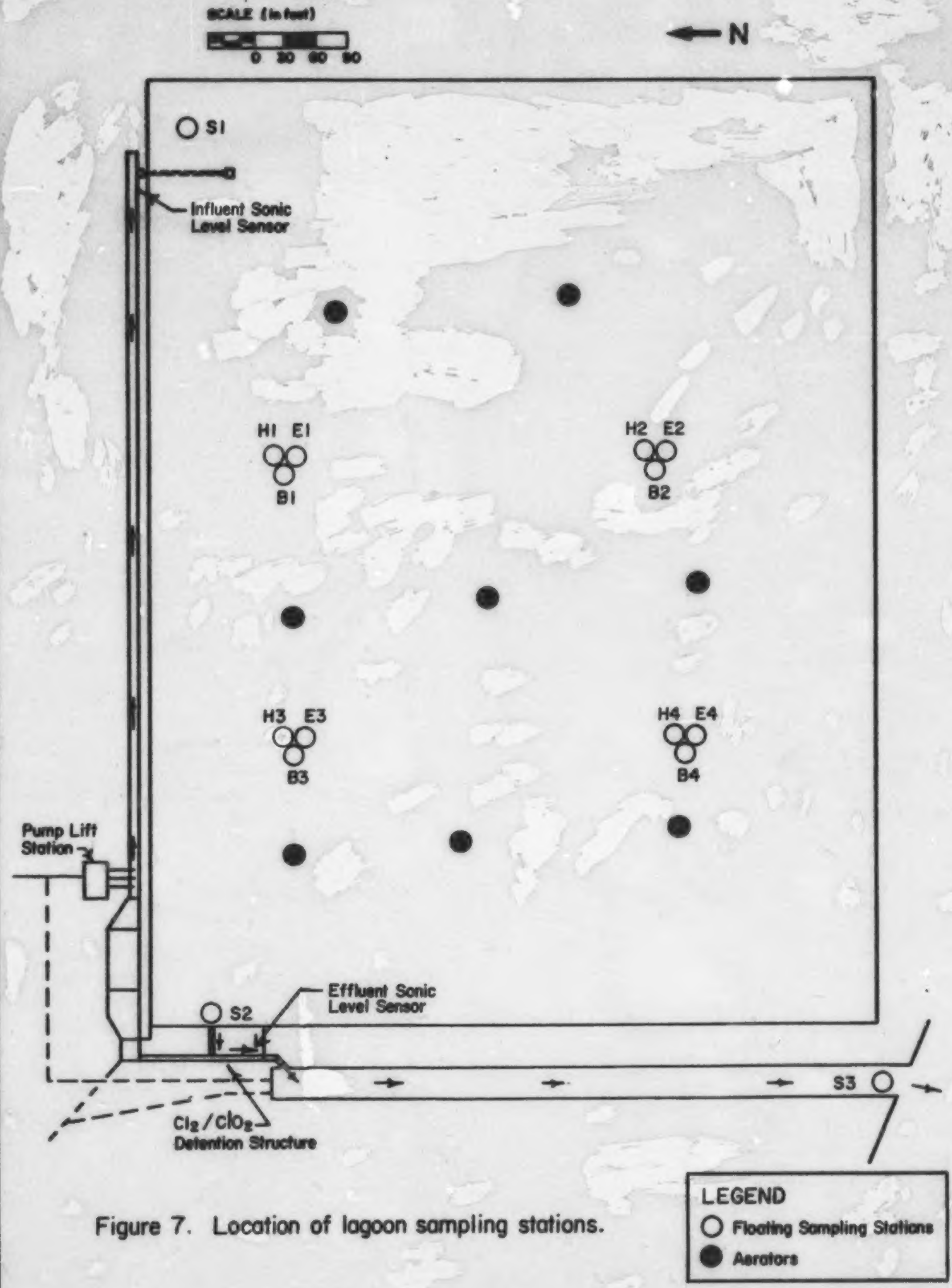

SCALE (in feet)
0 30 60 90
N
S1
Influent Sonic Level Sensor
H1 E1
B1
H2 E2
B2
H3 E3
B3
H4 E4
B4
Pump Lift Station
S2
Effluent Sonic Level Sensor
Cl_2/ClO_2 Detention Structure
S3
LEGEND
○ Floating Sampling Stations
● Aerators

Figure 7. Location of lagoon sampling stations.

5.3 Sample Collection Methods

Variations in the type, volume and analytical requirements of samples taken neccessitated the use of several different sampling devices. The need for representative influent and effluent samples required the use of composite samplers. Manning Samplers (Model 100) were installed at the desired locations, and adjusted to collect 1/2 liter samples at hourly intervals. Grab samples were collected in wide-mouth 1-liter plastic bottles when occasional automatic sampler malfunctions occurred.

The twelve stations within the lagoon were sampled on a daily basis. A one-liter plastic bottle was attached to a 3-foot steel pipe of sufficient weight to sink the assembly to the desired depth. Time of drop was minimal in comparison to bottle fill time. Accurate depths for each sample were determined by premeasuring the lines attached to the weight assembly.

Post chlorination samples for bacterial examination were collected as typical grab samples with the exception that they were immediately fixed with sodium thiosulfate to remove residual chlorine. Specialized studies such as dissolved oxygen and dye tracer evaluations were performed using the 32 station configuration. Dissolved oxygen and temperature profiles were performed using the appropriate probes.

5.4 Flow Gaging Equipment

Detailed hydraulic calibrations of the influent and effluent channels under various hydraulic conditions were performed to supplement the daily flow monitoring records and to determine performance reliability on the permanent ultrasonic flow meters. Daily records of flow depths were maintained and velocity readings at five points across each channel at the 0.6 depth level were measured using a Mead, Model MP 302 impeller-type velocity probe.

Specialized studies for developing stage/discharge relationships in each channel were performed using a Marsh-McBirney velocity probe. This electromagnetic device operates on the principle of noting magnetic flux changes with changes in water velocity. The unit contains no moving parts and suffers no interference from water-borne solids. Lagoon influent and effluent rates seldom contained solids concentrations sufficient to interfere with the Mead impeller meter, yet improved accuracy and reliability of the Marsh-McBirney meter justified its use for channel flow calibrations.

Flow rates within the influent and effluent channels, through the disinfection contact chamber, and along the earthen canal to the Grand Calumet River were measured using dye tracer methods. Rhodamine B, a red biodegradable dye, was used.

5.5 Lagoon Mixing Evaluation Methods

Lagoon mixing characteristics were expected to be directly related to influent flow rates and the positions of operating aerators. Dye tracer studies were used to monitor lagoon flow and mixing patterns under several aeration schemes. Dye concentrations were determined using a Turner fluorometer, Model III.

Dye tracer studies included visual monitoring of surface water dye appearance and spatial distribution in addition to vertical grab sampling. During the initial effort, samples were taken 1 foot below the surface and at the 15-foot depth at 32 locations within the lagoon and at the effluent. In later studies sampling was performed at the one-foot, ten-foot and 25-foot depths at 20 locations and at the effluent. Sampling sequences were established such that each station was sampled several times over the course of an experiment permitting interpretation of the migration of the dye throughout the lagoon. This routine required rapid circuiting of the stations near the influent station and end decreasing progressively toward the effluent stations. Usually after several hours the highest frequency of sampling was near the effluent end of the lagoon, followed by a final complete traverse of all stations. An additional profile was taken one day after each dye tracer study to verify the completion of mixing. Surface samples at the one-foot depth were collected by hand-held bottles whereas collection performed at the 10,15, and 25-foot depths utilized the apparatus set up for daily grab sampling.

5.6 Dissolved Oxygen Evaluation Methods

Several dissolved oxygen and temperature profiles of the lagoon were performed to complement daily records of oxygen depletion and replenishment. The initial profile covered 120 sampling sites, each measured at three depths. Due to the minimal variation of results, the number of sites was reduced to 32 sites, using the same locations as mentioned for the dye tracer lagoon mixing evaluations.

A boat was required for data collection. However, it was found that motor operation interfered with the dissolved oxygen determinations. The boat was thus intermittently rowed to monitoring locations and anchored.

A Yellow Springs Instrument Co. Model 57 dissolved oxygen meter was used, with a combination YSI model 5737 dissolved oxygen membrane probe and temperature thermister. The probe was calibrated by the saturated air method at the start of each profile and rechecked several times during the data collection process. (2) Additional calibration was performed by collecting water samples and using the Azide Modification of the Winkler Dissolved Oxygen test. (3)

The probe was attached to a steel pipe and lowered to the desired depth with polypropylene rope. The pipe apparatus was prepared such that when the lower end of the weight rested on the bottom, the probe would be approximately one foot above the bottom. During the reading of dissolved oxygen level, the probe was agitated by alternately raising and lowering the apparatus a few inches. This provided the fresh water film across the membrane necessary for accurate concentration determination.

5.7 Water Quality Parameter Coverage and Methods

Extensive analyses were performed to evaluate the pollutant removal effectiveness of the deep lagoon treatment. The frequency of the routine sampling and parameter coverage is indicated in Table 1. In general, the overall lagoon performance was evaluated by samples collected every other day from the influent and effluent and following any major storm event. Selected constituents were analyzed either four times per month or once per month. All analytical procedures were conducted by recently developed methods utilizing convenient equipment and supplies, furnished in the U.S. EPA furnished mobile laboratory. These methods are summarized in Table 2.

TABLE 1. SUMMARY OF MONITORING PARAMETERS/FREQUENCY

	B4	B3	B2	B1	E4	E3	E2	E1	H4	H3	H2	H1	S3	S2	S1	
pH in Benthos to be run 4 times per month Analysis every day	X	X	X	X										X	X	pH
BOD_5 every day for first two weeks then every other day for first month. Months 2-12 S1 & S2 every other day. B1-B4 run once a month	X	X	X	X	X	X	X	X	X	X	X	X		X	X	BOD_5
Analysis every other day					X	X	X	X	X	X	X	X		X	X	DO
Analysis every other day					X	X	X	X	X	X	X	X		X	X	TEMP
Analysis every day for first two weeks then every other day thereafter														X	X	TURB.
Analysis every day for first two weeks then every other day thereafter B1-B4 run 4 times a month	X	X	X	X	X	X	X	X	X	X	X	X		X	X	TSS
Analysis every day for first two weeks then every other day thereafter B1-B4 run 4 times a month	X	X	X	X	X	X	X	X	X	X	X	X		X	X	VSS
Analysis every day for first two weeks then every other day thereafter B1-B4 run 4 times a month	X	X	X	X										X	X	TS
Analysis every day for first two weeks then every other day thereafter B1-B4 run 4 times a month	X	X	X	X	X	X	X	X	X	X	X	X		X	X	NO_3^-
Analysis every day for first two weeks then every other day thereafter B1-B4 run 4 times a month	X	X	X	X	X	X	X	X	X	X	X	X		X	X	P
Analysis every day for first two weeks then every other day thereafter B1-B4 run 4 times a month	X	X	X	X	X	X	X	X	X	X	X	X		X	X	COD
Analysis every day for first two weeks then every other day thereafter B1-B4 run 4 times a month	X	X	X	X	X	X	X	X	X	X	X	X		X	X	TOC
Analysis every other day													X	X		F.COL
Analysis every other day													X	X		T.COL
Analysis every other day													X	X		F.STRP
Analysis every other day													X			Cl_2
Analysis every other day B1 & B3 run 4 times a month		X		X										X	X	Fe
Analysis every other day B1 & B3 run 4 times a month		X		X										X	X	Cd
Analysis every other day B1 & B3 run 4 times a month		X		X										X	X	Cr
Analysis every other day B1 & B3 run 4 times a month		X		X										X	X	Cu
Analysis every other day B1 & B3 run 4 times a month		X		X										X	X	Pb
Analysis every other day B1 & B3 run 4 times a month		X		X										X	X	Zn
nalysis every other day 1 & B3 run 4 times a month		X		X										X	X	Hg

S1 - S3 - Surface
H1 - H4 - Hypolimnion @ 25 feet
E1 - E4 - Epilimnion @ 10 feet
B1 - B4 - Benthic Layer

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD, BEGIN SECTIONS HERE

TABLE 2

SUMMARY OF ANALYTICAL LABORATORY METHODS

Parameter	Technique
Dissolved oxygen	YSI Membrane Electrode; Azide Modification of Winkler Test (3)
pH	Orion specific ion electrode
Temperature	YSI electrode with thermister
Turbidity	Hellige Corp. Turbidmeter
Solids	SM 208AE*
BOD	Respirometer by Oceanographic International (4) and SM507*
COD	Automatic Ampule Method-Oceanographic International (5)
TOC	Beckman 915A TOC Anlayzer with printer
Fe,Cd,Cr,Cu, Pb,Zn and Hg	Atomic Absorption SM*
Fecal Coliform, Total Coliform, and Fecal Streptococcus	Membrane Filter Methods SM 909C, 909A, 910B*
Chlorine/Chlorine Dioxide Residuals	Amperometric Titration SM 409C, 411C*
Nitrates	Orion Specific Ion Probe
Phosphorus	Ascorbic acid method SM 425 F*

---

* Standard Methods (3).

6-1/2"
9-1/8"
BEGIN LAST LINE OF TEXT
3/8"
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS

EPA-287 (Cin.)
(4-76)

CHAPTER 6

CALIBRATION OF LAGOON APPURTENANCES

6.1 Foreword

A discussion of the calibration procedures used to determine the discharge rates of the dry weather flow lift station pumps is presented in section 6.2. Discussions of the calibration procedures and results for developing the stage/discharge relationships used to estimate lagoon influent and effluent rates are presented in sections 6.3 and 6.4, respectively. An analysis of chlorination chamber detention periods later used in the disinfection experiments is presented in section 6.5. An analysis of exfiltration rates from the lagoon is presented in section 6.6. Calibration details of the various components used in the sequential chlorine/chlorine dioxide disinfection system are presented in section 6.7.

6.2 Calibration of Lagoon Influent Pumps

Three pumping units continuously pump secondary treatment effluent to the lagoon. The pumps, distinguished by a numbering system (see Figure 2), include one variable speed Cascade pump (No.1), one Submersible Flygt pump (No. 2), and one constant speed Cascade pump (No. 3). The pump rates were unknown prior to this study and calibration was performed using velocity measurements and dye studies at several "pump on" combinations. During these studies, it was noted that any given "pump on" combination could yield varying lagoon inflow rates.

The resultant pump capacities were determined as 7.8 mgd (0.34 cms) for pump #1, the variable speed pump operating at 90% speed; 3.1 mgd (0.14 cms) for the #2 pump and, 8.2 mgd (0.36 cms) for the #3 pump operated at its fixed speed of 94% capacity. These values were determined for total discharge heads of about 30 feet (9.2 m) with the sump water surface about 6 inches below the sump walkway. Higher values were recorded when the pumps were individually tested in comparison to "all on" combinations. These increases were due to the additional head deriving from the sump water level rise during the period of testing at less than "all on" conditions. Considerable difficulty was encountered in calbrating the pumps due to a number of factors including, extremely variable plant inflow, the fixed speed mode of pump operation and flow reversal in bypass pipes (pumpage from the river). For sake of completeness a discussion of these problems will follow.

The influent discharge at the ECSD WWTP averages 18 mgd (0.79 cms) and cyclically varies by 2 to 4 mgd (0.09-0.18 cms) on a 30 to 40 minute basis with irregular variations of about 4 mgd. In addition, day/night variation is about 5 mgd (0.22 cms). Although the influent to the lagoon is supplied by constant speed pumps, this constantly varying plant discharge changes the water surface (head) in the wet well resulting in considerable variation of pumpage rates.

The configuration of the pumping system is such that excess plant flow above the lagoon pump capacity at any given head is bypassed to the Grand Calumet River. In addition, plant flow rates insufficient to meet the pump capacity create a reversal of bypass flow such that river water is drawn into the system and is pumped into the lagoon. The actual water level in the pump sump is a function of the rate of change of plant flow, the rate of flow between the pump sump and the river, and the actual pump rate. The plant inflow is subject to the flow changes discussed above which continue through the WWTP with little dampening. The rate of pumping is subject to the change in water surface level as it affects the total dynamic head. Flow between the pump sump and the river is determined by the level of the river water, the difference in the water levels between the river and the sump and the extent of the gate opening between the two locations. The net result is a continual variation in the sump water level as the pumps constantly seek equilibrium with the river, while being subject to short-term inflow rate changes.

Two consequences of this flow pattern configuration result. First, the lagoon inflow rate changes as a function of the varying total head applied to the pumps. From the manufacturer's head/discharge curve, a change in water depth of 4 inches would change the flow rate by about .20 mgd for each of the two large pumps. A water depth record during the calibration period for the lagoon influent channel showed the cyclic inflow change to be about .45 mgd when the 2 large pumps were on. With two pumps on the return to equilibrium level would be slower after sudden flow surges. Greater head variations would be expected than for during operations with all pumps on.

During the calibration period, the treatment plant flow varied from 11 to 17 mgd, and the sluice gate between the pump sump and the river was completely open. Normally, the flow rates range from less than 10 to greater than 20 mgd and the sluice gate is typically left in a semi-closed position. Partial closure of the sluice gate would result in greater head changes on the pumps with a resultant increase in lagoon inflow variation.

The end result is that at any time the lagoon inflow rate will vary under any "pump-on" combination selected at the plant. The 8/19/79 influent calibration indicated a change in inflow rate of about 2 mgd over a one hour period with all pumps on. Inflow calibrations at other times indicated similar variations using different pump combinations.

The second effect of this flow scheme is the change in water depth at the bypass discharge pipe to the Grand Calumet River. Current practice is to allow the Grand Calumet River to act as a buffer to pumping operations. When flow from WWTP exceeds pump capacity, the lagoon is bypassed. When the WWTP flow is less than the pump capacity a flow reversal occurs in which river water flows back into the bypass pipe to the sump well in sufficient quantity to equal the difference between the pump rate and the plant flow.

Given that the pumping rate is subject to the aforementioned problems, the only action available to control the pumping rate are the combinations of "pumps on" and the head supplied. In the case of high treatment plant flows, all pumps would be operating and control could only be achieved by varying the position of the bypass sluice gate. At present, this gate is only manually operated. Complete corrective action would require a semi-automated system wherein the treatment plant flow rate is related to the pump sump water level and inter-connected to regulate the opening and closing of the sluice gate, and the selection of "pumps on" combinations.

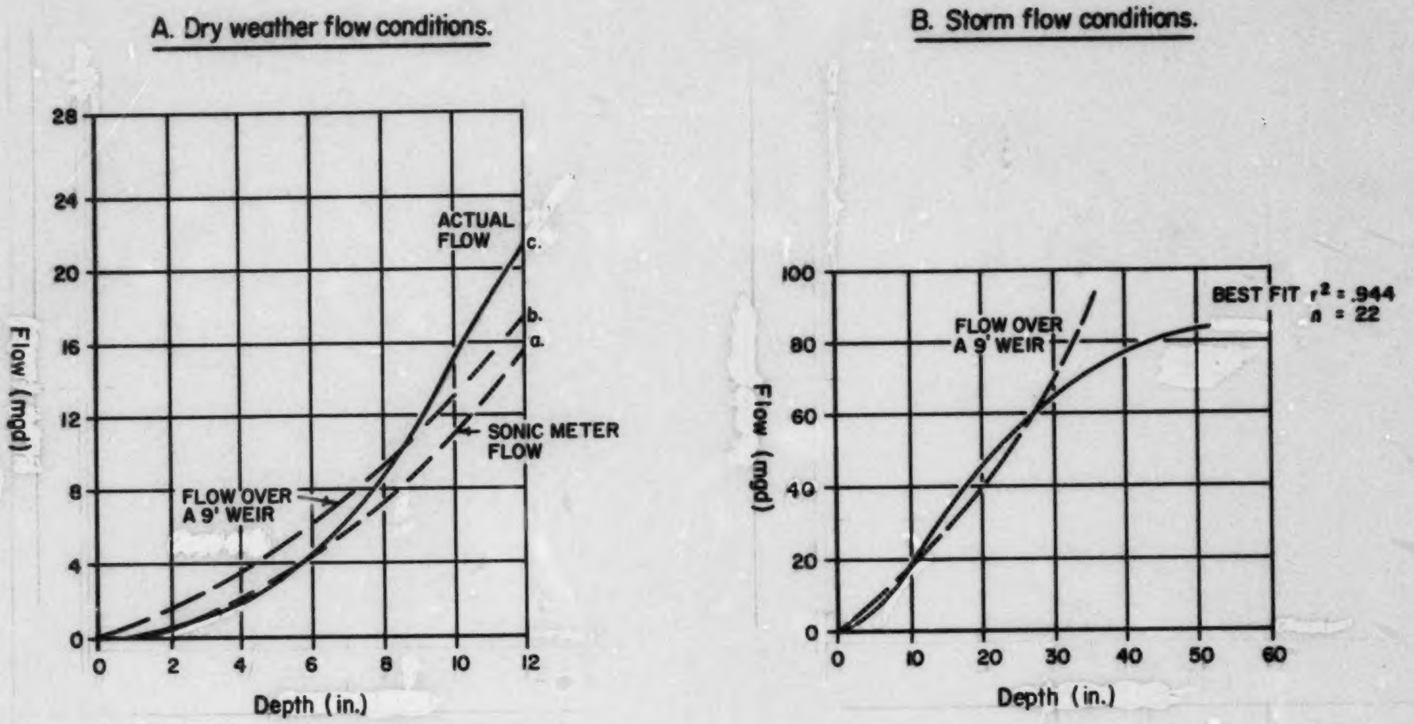
6.3 Influent Channel Stage/Discharge Relationships

Influent channel flow rates were monitored by a Manning ultrasonic meter which interprets the inflow water surface as the height above a 9-foot wide rectangular weir. This value is electronically converted to a percentage of 30 mgd as indicated by a pointer meter and is also permanently recorded on polar charts.

Due to flow turbulence and bottom irregularities at concrete expansion joints, wave action within the influent channel necessitated the use of a dampening device to eliminate recorder response to instantaneous changes. Since this correction was made, precise depth determinations were recorded.

Numerous pump combinations were selected to provide discharge into the channel at varying flow rates for the dual purpose of determining pump capacities and to develop an influent channel stage/discharge relationship. The results of these analyses are shown in Figure 8. Various stage/discharge relationships for dry weather pumped discharge conditions are shown in Figure 8A while an extended relationship for storm water inflow conditions is depicted in Figure 8-B.

Curve "a" in Figure 8A was established by ECSD prior to this project. Curve "b" in Figure 8A is a theoretical curve based on a 9-foot broad crested weir. Curve "c" in Figure 8-A was developed from 14 measured stage/discharge points evenly distributed with depth of flow. A channel bottom irregularity



A. Dry weather flow conditions.

B. Storm flow conditions.

Figure 8. Stage/discharge curves, influent channel.

creating a one-inch deep puddle of water was discovered during a period of no influent flow at the location of the ultrasonic meter. Much of the pre-project controversy over the actual rate of the 3mgd submersible pump in the lift station wet well stemmed from the inaccurate rating curve.

Twenty two flow calibration points used to derive the curve in Figure 8 were collected during storm events. The curve shown accurately represents these data and indicates the effect of the limiting discharge condition of the 48 inch pipe outlet from the channel into the lagoon. Various stage/discharge relationships for dry weather pumped discharge conditions are shown in Figure 8A while an extended relationship for storm water inflow conditions is depicted in Figure 8. For general purposes, the flow predicted by thhe 9-foot weir equation can be considered acceptable at rates less than 60 mgd.

6.4 Effluent Stage/Discharge Relationships

During the evaluation period, lagoon effluent flow rates were to have been monitored by a Manning ultrasonic flow meter and permanently recorded on polar charts. Delayed installation, occasional malfunctions, and the lack of a dampening circuit within the device rendered this means of flow monitoring to be unreliable. Much of the disinfection experimental program was conducted using erroneous discharge information from the ultrasonic meter. The erroneous measurements were detected late in the study and prompted a detailed effort to develop alternative flow calibration curves. The program designed to measure lagoon effluent flow rates for occasional calibration of the meter became a major concern of the project.

The lagoon effluent spillway is characterized by a dry weather channel cut into a wet weather apron (see Figure 5-A). Under typical flow rates, all effluent water is contained within this channel, which leads to the disinfection chamber. The disinfection chamber consists of a 6-foot wide concrete channel with 10-foot high side walls and a 3-foot steel baffle at its end acting as a rectangular weir without end contractions. Effluent water falls up to 7 feet between the spillway and chamber. The resulting turbulence necessitated the use of a wave buffering plate between the falls and the weir, to facilitate the operation of the automatic flow measuring device located above the weir. Due to the limited capability of this plate to buffer all surface turbulence, a dampening circuit within the meter was required to eliminate recorder fluctuations due to instantaneous surface level changes.

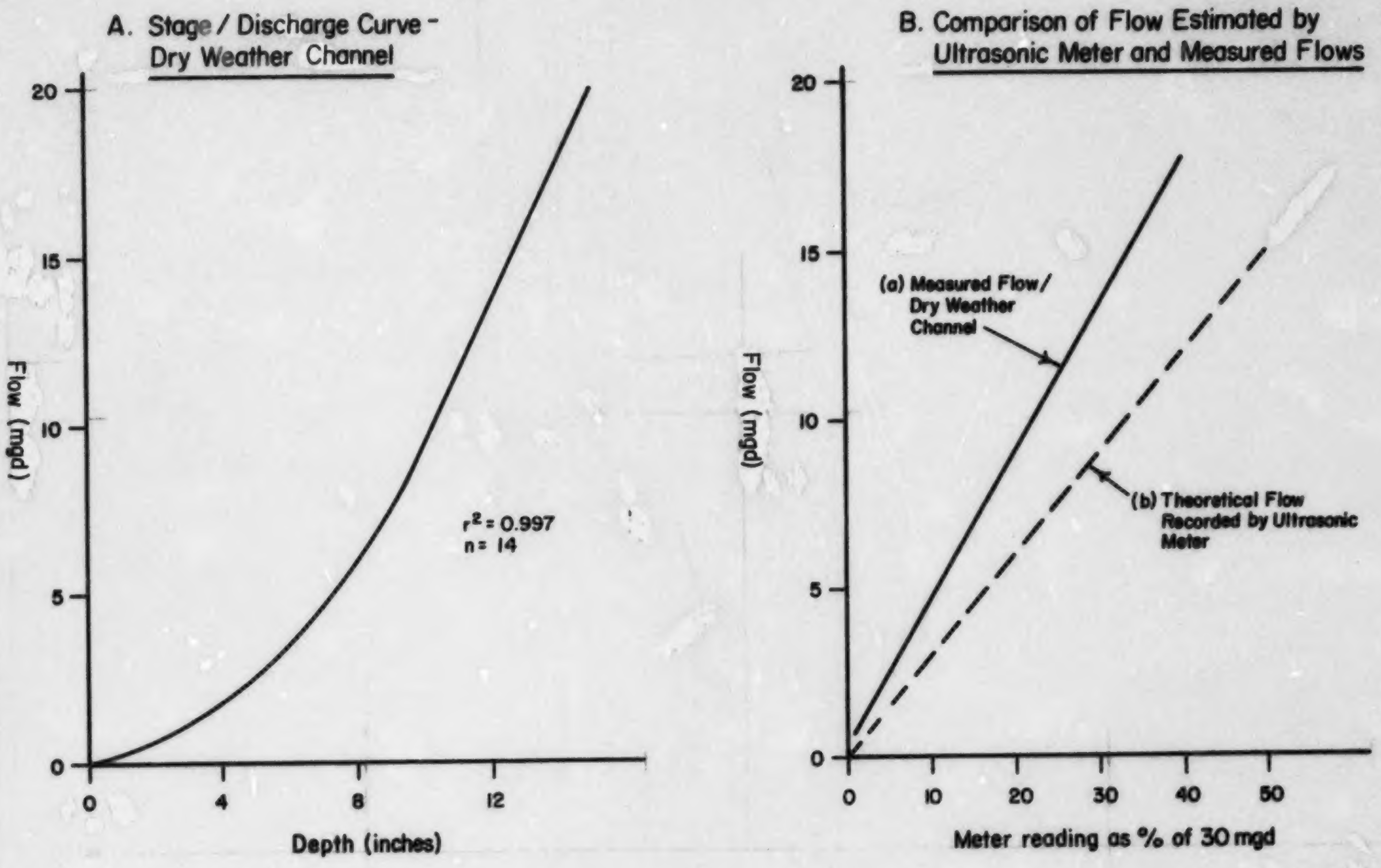
Effluent flow rates were manually monitored within the dry weather spillway. Five locations across the 4.5 foot channel were each evaluated at 2 to 3 depths, depending on the depth of flow which ranged from several inches to over 1 foot in the 24 inch deep channel. Flows were determined based on the cross-sectional area and average flow rate recorded. Although the ultrasonic meter is located in the lower chamber, earlier attempts to calibrate flow in this section proved futile due to high turbulence causing constant fluctuation in velocity and direction of flow at any point. Flow in this channel was, therefore, recorded on a regular basis as a depth measurement only. This value was then adjusted to a respective height over the weir and also referenced to the flow manually measured. A single attempt was made to determine the flow rate by velocity measurement at the weir crest where flow is also laminar. Measurements indicated a rate of flow equivalent to that determined in the dry weather spillway. Although results were good, this location was very difficult to properly analyze and flow monitoring was limited to the flow determination in the dry weather spillway, and depth measurements in the disinfection chamber as stated above.

Results of the dry weather channel measurement program were summarized and evaluated to produce a stage/discharge relationship shown in Figure 9-A. Within the dry weather spillway, rate of flow was determined to be precisely related to the depth. Recorded data for this curve showed little variation from those predicted by the best fit curve at any given flow as indicated by the high correlation coefficient (r^2=0.997,n=14).

Although the initial ultrasonic meter installation produced unsteady output, the addition of a dampening circuit reduced this variation such that reasonable records were produced. This unit was set to record the effluent flow rate as a percentage of 30 mgd based on the height of water over the crest of a 6-foot rectangular weir. Daily records were made using the meter along with simultaneous manual water surface level determinations. Actual effluent flow rates as determined in the dry weather spillway, are equated to corresponding ultrasonic meter records in Figure 9-B. The actual rate clearly differs from the predicted rate of flow, indicated by the dashed line. The degree of fit for the data gathered to develop the curves in Figure 9-B was high (r^2=0.996 n=10).

Flow data was next evaluated by determining actual heights over the weir based on the surface level records and comparison to known dry weather channel rates. Results of this analysis are shown in curve "a" in Figure 10A. Measurements for surface levels were made manually. As expected, the same level fluctuations which inhibited ultrasonic meter record evaluation, limited the accuracy of manual measurements, as reflected in the somewhat lower correlation coefficient of 0.917 (n=33).

Theoretical estimates of flow over a properly constructed and installed weir is also shown as curve "b", Figure 10-A. It can be seen that there are significant differences



A. Stage / Discharge Curve - Dry Weather Channel

B. Comparison of Flow Estimated by Ultrasonic Meter and Measured Flows

Figure 9. Stage / discharge curves, lagoon effluent.

between curves "a" and "b". The problem can be assumed to encompass more than simple miscalibration of equipment since the discrepancy involved not only flow differences but also overall curve shapes.

A review of standard weir installation practice reveals several areas of deficiency within the East Chicago installation. Optimum weir performance requires strict adherence to numerous installation guidelines including, a sharp edge at the weir crest with right-angled corners, specific edge thickness or chambered-edge construction, a smooth and straight front surface perpendicular to the direction of flow, a means of removing accumulations of settled debris from the area upstream of the weir and proper ventilation of the weir nappe. Observation of the steel plate installed in the effluent chamber showed this device to be of an overall thickness greater than recommended to be used without a chambered trailing edge. The weir resembles a broad crested weir rather than a sharp crested unit, for which the recommended procedure is in-situ calibration. In addition, the leading edge was somewhat eroded, providing less than the required edge sharpness which can decrease head at a given flow rate. Little accumulation of debris was found during the study which mitigated the need for the recommended removal mechanism. Located only several feet upstream of the weir are three chlorine diffusion devices used with the pre-project disinfection system. These devices are simply steel pipes located at the bottom of the contact chamber. The effect of these devices as flow obstructions could not be determined due to the depth and turbidity of the effluent waters.

The recommended weir approach consisting of an unobstructed section of flat slope is 25 feet under maximum pumping conditions and 35 feet at the maximum sonic meter reading of 30 mgd. Approximately 23 feet upstream of the weir a wave baffle was installed to limit surface turbulance to enhance surface level determination. Visual observations showed that this device created turbulent flow, but the distance to the weir was sufficient for transition to laminar flow.

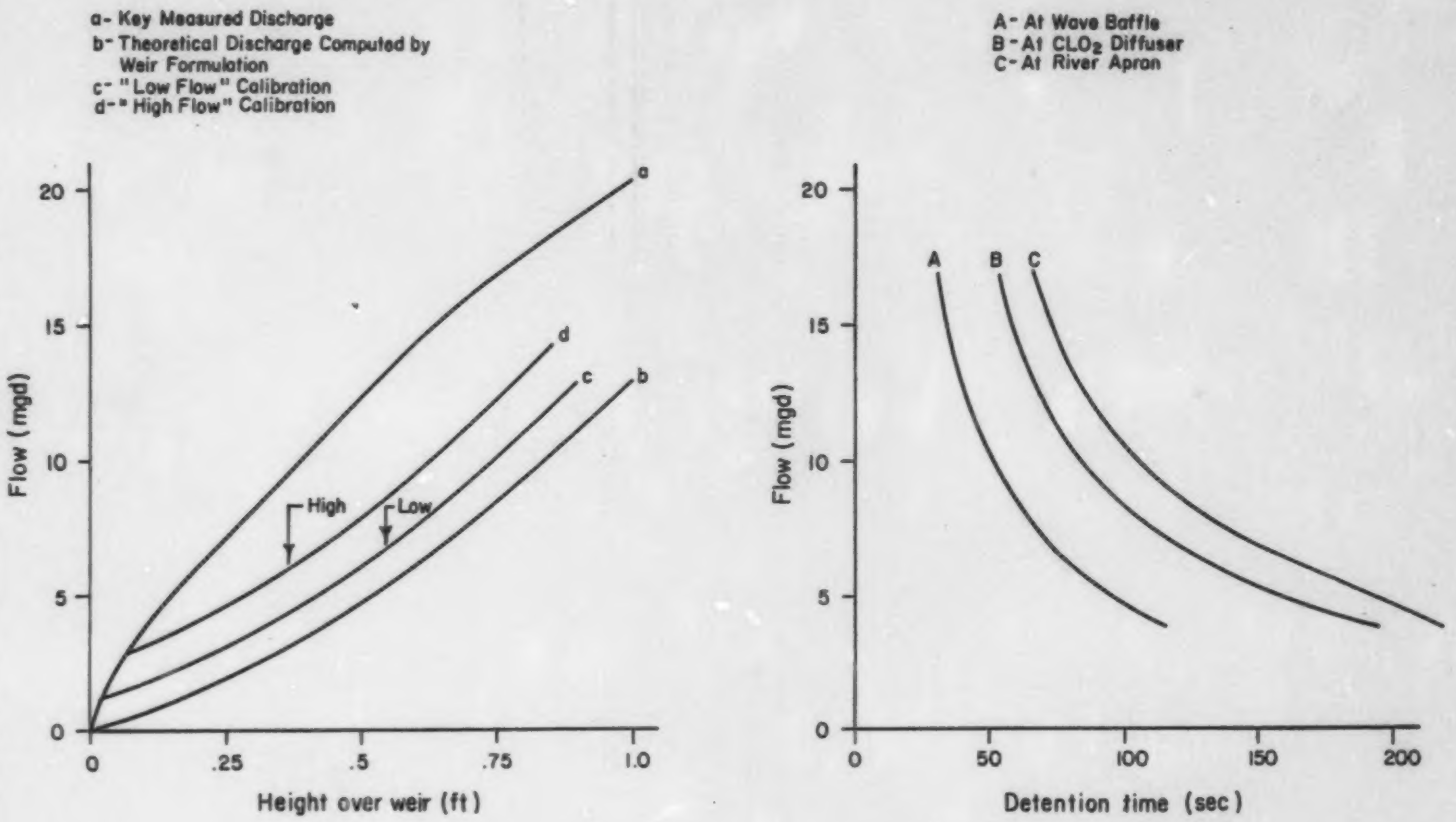
Inspection of the channel sides at the weir and immediately downstream showed definite interference with proper weir operation. The weir is embedded into concrete walls on each side. One side is straight after the weir, whereas the opposite side angles inward, restricting the nappe flow. This projection appears not to influence the flow over the weir crest itself, and the effect of nappe deformation is unknown. The ineffectiveness of this configuration is in the formation of a vacuum under the nappe which reduces overall head. The effect of improper ventilation was noted late in the evaluation program. A small diameter steel pipe was inserted through the nappe to the underlying air space and created an immediate response in flow characteristics. The nappe propelled further out from the weir and substantial rise in head over the weir occured. The pipe itself did not interfere with the flow, since upon removal, flow conditions remained temporarily unchanged. As time passed, vacuum conditions again set in with eventual return to the preventilated nappe configuration.

In light of all these variables, the difference in theoretical and actual flow rates reported in Figure 10-A are quite realistic. The question arises whether rehabilitation of this measuring configuration is necessary. Many of the deficiencies could easily be corrected including weir crest design, nappe ventilation and removal of the diffuser pipes. Further corrective action could be quite complicated. Preventing nappe constriction would require complete movement of the weir upstream further from the side wall contraction. This would reduce the overall volume of the disinfection contact chamber, reducing detention time and thus disinfection effectiveness. The weir would also be closer to the wave baffle and have less opportunity for transition to laminar flow.

Assuming all these corrective actions were taken, it would still be necessary to cross-check new flow rates determined against theoretical values. It would seem quite unlikely that flows would be exactly as theoretically predicted and formation of a calibration curve would again be necessary. The calibration curve developed using measured dry weather channel flows for relating flow to weir crest depth is adequate.

Although a large discrepancy exists between actual flow rates measured and those theoretically predicted, the cumulative effects of improper weir installation techniques justify the observed diminished weir heights. Corrective actions are unnecessary in light of the good correlation of flow to depth readings presented. It is therefore recommended that no changes be made to the weir or surrounding components. Flow records should be either adjusted manually using the relationship developed here and presented in Figure 10-A or the metering devices be electronically restructured to translate data using the relationship developed and prescribed in Figure 9-B.

One further note concerning the effluent monitoring device was the flow calibration efforts conducted early in the project. During the initial phase of the project the effluent device was calibrated at a known flow rate of 3 mgd by others without consideration to the actual height over the weir and theoretical flow rate. This resulted in the "low flow rate" curve "c" of Figure 10-A. The ultrasonic meter was then recalibrated by others during latter portion of the evaluation period when high discharge rates were used and yielded results exactly 2/3rds the actual values at all velocities. The "high flow rate" curve is



A. Various Stage/Discharge Curves- Effluent Weir.

B. Detention Time/Discharge Curves at Chlorination Chamber.

Figure 10. Stage/Discharge curves at effluent weir and detention time/discharge curves at chlorination chamber.

presented as curve "d", Figure 10-A and was then used during much of the lagoon flow analysis.

Had the initial calibration included a cross-check with the theoretical flow value, the variance discussed herein would have been discovered at the onset and corrective actions taken. The lack of this correlation led to improper automatic flow recording throughout the study, and strengthened the erroneous belief in excessive exfiltration rates. The importance in frequent calibration is well understood and the point here is that this process should include more than setting or resetting a measuring device to match manually determined flow rates. Comparison of head measurements to flow rates must be performed to ensure that actual differences are meter-related and not due to improper construction or time-induced changes in flow chamber configuration.

6.5 Analysis of Chlorination Chamber

Flow rate calibration of the lagoon effluent included velocity measurements and conducting dye studies. The use of dye studies enabled the additional determination of average detention time through the disinfectant contact chamber. Three locations were evaluated for detention time at different rates of flow including, the wave baffle assembly, the ClO_2 diffuser and the end of the apron at the overflow to the river. The reference point for the three locations is the Cl_2 diffuser located at the bottom of the dry weather spillway. Detention time versus discharge curves for the three locations are presented in Figure 10-B.

6.6 Exfiltration Rates

During construction of the lagoon, it was decided to use the materials removed from the basin area of the lagoon to construct the surrounding levees. The materials used were from an abandoned sanitary landfill. The quality of this material for levee construction has been continually questioned, and the extent of exfiltration estimated on several occasions. It was conjectured that due to the lengthy period since construction, exfiltration through any porous sections would have deposited sufficient solids in the voids to impede further loss. This study determined the actual rate of exfiltration by closely monitoring the influent and effluent flow rates and the evaporation rate from the lagoon surface.

During a period of four consecutive weeks, influent and effluent flow rates were monitored several times daily and the consistency of the ultrasonic meters verified. These meters produce permanent flow records on polar charts operating on a seven-day basis. The resolution of these charts allows determination of flow rates to within about 0.1 mgd. Small changes in water depths due to turbulence and wind effects occasionally caused instantaneous static lines on the permanent charts necessitating an averaging procedure during interpretation. Initial attempts were made to evaluate the influent and effluent flows more closely with more sophisticated depth monitoring equipment utilizing 24-hour charts and expanded deflection ranges. It was found that the water surface level changes previously mentioned in Section 6.2 caused too great an interference with recorder measurements to yield improved precision over the standard sonic meters in regular use. It was then decided to use the ultrasonic meter 7-day charts for exfiltration evaluation.

Exfiltration rates were determined by a flow balance of all means of lagoon water exchange, including influent and effluent flows, evaporation and precipitation. Four consecutive seven-day charts were analyzed to determine the average flow rate over a 28-day period during which there was no precipitation. Evaporation rates were determined by recording daily change in depth of lagoon waters within a plastic bucket partially immersed within the lagoon. Rates of evaporation showed little daily variation coinciding with the mild even temperature encountered during this period. Average water loss from the lagoon surface by evaporation was calculated to be 0.1 mgd.

Each of the influent and effluent charts were analyzed to determine the average weekly inflow and outflow rate. Since each chart contained some areas of poor resolution due to instantaneous depth changes, maximum and minimum flows were determined and averaged. These estimates were then averaged over the entire four week period.

The average recorded influent flow was determined to be 11.4 mgd. The average effluent was 11.3 mgd. Considering the total lack of precipitation during this period and that a summation of the remaining peak flow spikes amounted to a negligible quantity, it would appear that recent reports of high rate exfiltration were in error due to the misrepresentation of the sonic flow meters. Although in years past more specific analyses were conducted reporting low-level exfiltration rates, the results of this study indicate that, as predicted during original construction, settled solids had successfully filled void spaces.

6.7 Calibration of the Disinfection System

Prior to evaluation of the two-stage chlorine/chlorine dioxide technique as a disinfection mechanism, it was deemed valuable to calibrate components of the system and optimize chemical requirements for chlorine dioxide production.

Flow meters within the system indicate rates of flow for chlorine feeding the chlorination system and the ClO_2 production system and the city water to the ClO_2 generator. Accuracy of the rotameters used for chlorine gas flows was deemed acceptable based solely on manufacturer's specifications.

The city waterline flow rate was desired for computation of actual ClO_2 production. The meter supplied with the system was of questionable accuracy. A simple procedure was found to measure the actual rate of flow. A city water reservoir tank is kept constantly overflowing by filling the tank slightly faster than the rate of pumping to the ClO_2 generator. By stopping the inflow to the reservoir tank and recording the rate of surface drop, the actual pump rate to the generator was determined.

Calibration of the chemical feed pump supplying the sodium chlorite solution was initially performed by detaching the chemical line to the generator and measuring the rate of flow at various settings. The chemical feed pump rate can be set by limiting the percentage of each stroke or changing the pump speed thereby regulating the stroke frequency.

During initial calibration, rates were determined as ml of sodium chlorite solution pumped per minute at two pump speeds over the entire range of stroke settings. These were found to vary somewhat from the rates originally determined by the disinfection system supplier.

It was noted during the calibration procedure that non-syphoning mechanisms were not incorporated into the system which would prevent chemical flow once the line feeding the generator was disconnected. During the initial calibration, the sodium chloride tank level was low enough so that the chemical line could be held above the equilibrium level during calibration.

At the time initial calibration was performed, disinfection dosages were set manually by adjusting feed rates to those specified by the chemical supplier to yield applied dosage of 8 mg/1 Cl_2 and 2 mg/1 ClO_2.

Original design of the disinfection system specified that dosages be automatically flow-proportioned to effluent rates. Automatic proportioning of the sodium chlorite solution was to have been accomplished by varying the pump speed. Although the system was never completely automated for flow proportioning during the evaluation period, it was desirable to have complete calibration curves available. A specialized technique was developed for pump rate evaluation such that the effects of backpressure in the chemical line developed at the generator would be included with this technique and the present calibration and future calibration, if warranted, could be performed without disassembly of the system.

Sodium chlorite solutions are extremely alkaline. A 1% solution has a pH of about 10.7 and this characterisic was used to evaluate pumping rates. A titration curve of the pH increase of city water with sodium chlorite solution addition was performed in the laboratory and is shown in Figure 11-A. Sodium chlorite pump calibration was then performed by discontinuing the chlorine gas flow to the generator and varying the pump settings and recording the pH of the aqueous $C10_2$ solution sampled after the generator. Knowing the previously calibrated city water flow rates were calculated using the information in Figure 11-A. The resulting sodium chlorite pump calibration curves are shown in Figure 11-B.

Complete calibration of the sodium chlorite pump covered the entire stroke and speed settings. Automation of the disinfection system will rely on a manually set pump stroke with pump speed automatically varied to yield flow proportioned to lagoon effluent rates. Optimum operation of the system would seem to imply setting the stroke at a constant value of 100 percent, allowing the initial speed setting to be as low as possible. This would yield the maximum available sodium chlorite level to be pumped during storm flow periods. Analysis of the generator effluent by pH techniques demonstrated that the slugs of percursor solution produced by high stroke-low speed pump settings are not completely equalized in the generator. Actual operation for chlorine dioxide production at these settings would yield inefficient chemical usage with alternative excess and insufficient percursor solution being applied. Based on this pump characteristic, it was found that speed settings below 20 percent should not be utilized.

The reaction of chlorine and sodium chlorite to produce chlorine dioxide requires one-half pound of chlorine to produce one pound of chlorine dioxide. Each pound of chlorine dioxide produced also requires one-half gallon of the percursor solution containing 25 percent available sodium chlorite.

Chlorine dioxide production was optimized by determining levels in the generator effluent while varying the chlorine gas addition at constant sodium chlorite rates. Determination of chlorine dioxide levels in the aqueous solution were performed by several techniques. The amperometric titration technique of Standard Methods (3) was used. A typical $C10_2$ production curve is shown in Figure 12. A field test for the determination of $C10_2$, developed by the chemical manufacturer, was also used due to its simplicity. The method required the addition of 1ml of buffer followed by a colorimetric titration using a dropper bottle. Each drop of titrating solution equaled 0.05 mg/l of chlorine dioxide as $C10_2$.

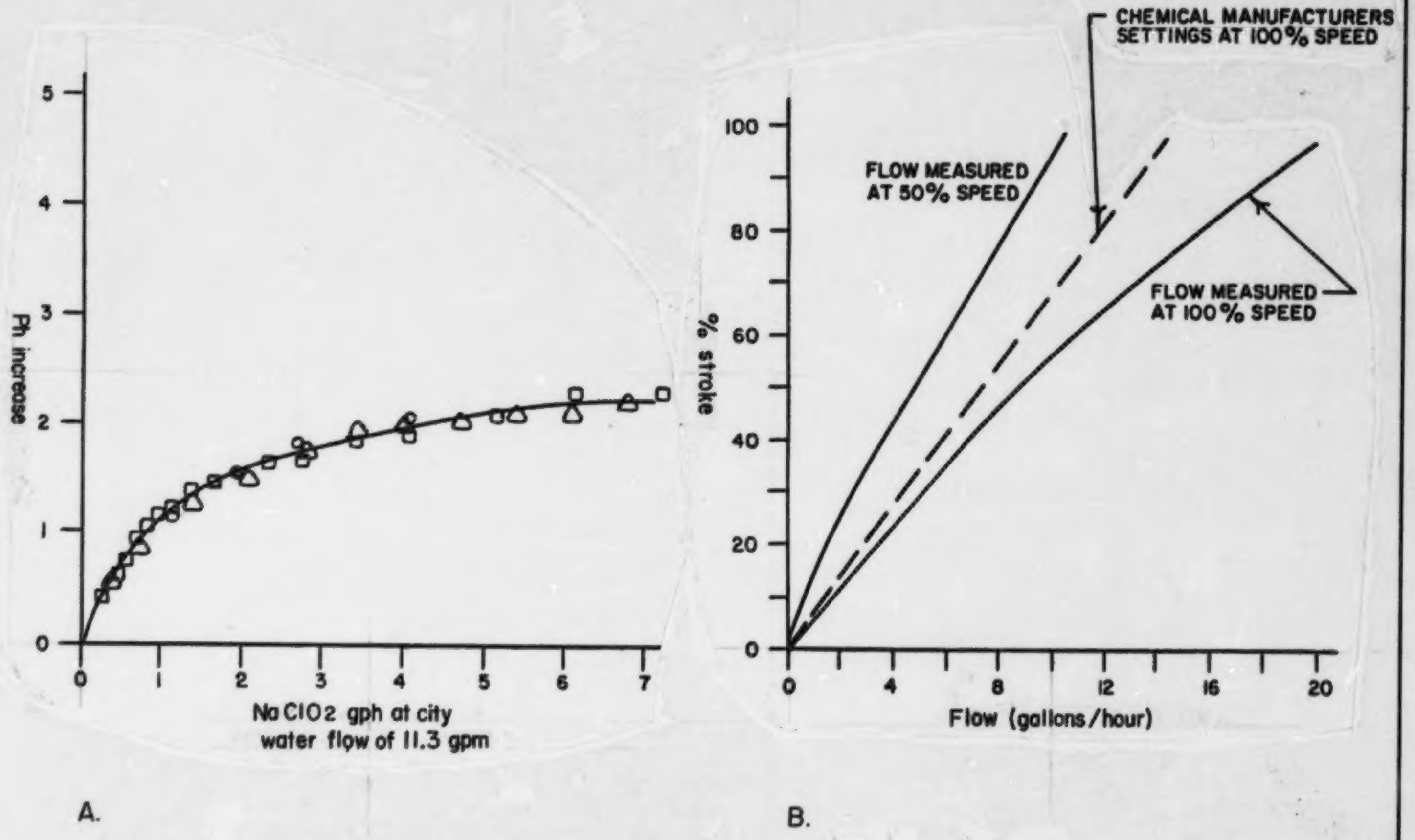


Figure 11. Calibration curves / disinfection system components.

Chlorine dioxide concentrations in the aqueous solution from the generator were in the ranges of 500-1000 mg/l. Each of the above techniques required dilutions of 500 to 1 or more, thus ClO_2 determinations varied slightly from the theoretical on several occasions. The simplified field technique was found to yield results about one-half the predicted theoretically and determined with the amperometric titration method.

During the optimization of ClO_2 production the amperometric titration method demonstrated production trends somewhat different than expected. Figure 12 indicates the effects of increasing Cl_2 addition. The actual Cl_2 requirement is seen to be always about 10 mg/l above the theoretical requirement. This requirement could result from initial chlorine demand of the carrier water or improper calibration of the chlorine feed system. Since the chlorine residual or demand of the city carrier water is specific to the East Chicago application, detailed analysis to determine this cause was not warranted, as the data would not be applicable at other locations.

Problems affecting optimization of ClO_2 production, as indicated by the amperometric titration method, occurred in the range of chlorine addition in excess of the theoretical requirement. At each sodium chlorite level, ClO_2 produced would peak and then decline sharply. Communication with the chemical manufacturer to explain this effect resulted in a third method for ClO_2 determination. Although yet to be proven valid, the method included spectrophotometric analysis of the ClO_2 sample. Results of these analyses are also shown on Figure 12.

The net result of the anlaysis is that optimum ClO_2 production requires Cl_2 addition slightly above the theoretical requirement at the East Chicago disinfection site. This result is based on the specific carrier water used in East Chicago and interpretation of the Cl_2 flow meters.

It was beyond the scope of this project to determine why excess chlorine supply interfered with analysis of ClO_2 residuals by the amperometric titration method. Both analytical techniques indicated the same optimum combination of chlorine and sodium chlorite for chlorine dioxide generation. Chlorine additions beyond this level would be inefficient and unnecessary with the control mechanisms available. Emphasis during the study was on optimum bacterial kills based on dosages of the sequential Cl_2 - ClO_2 system.

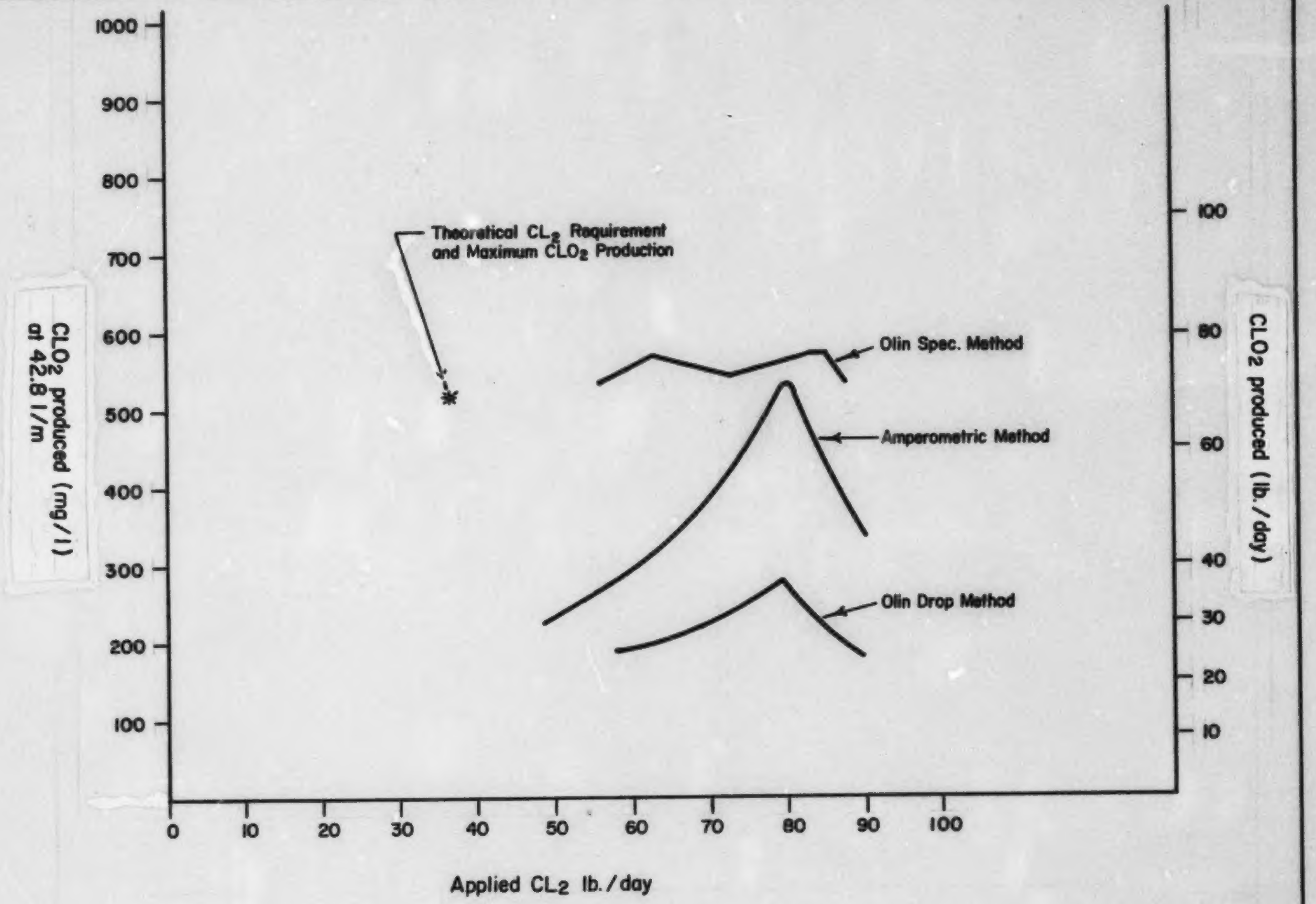


Figure 12. Chlorine dioxide production curves.

CHAPTER 7

ANALYSIS OF LAGOON MIXING CHARACTERISTICS

7.1 Foreword

Evaluation by dye tracer studies of the lagoon flow and mixing characteristics under different combinations of operational aerators is presented in section 7.2. Detailed velocity measurements of currents induced by a single operational aerator are presented in section 7.3. Dissolved oxygen profiles performed throughout the evaluation period are presented in section 7.4.

These physical measurements provided insights into the need and extent of the aerators to perform the dual function of ensuring adequate mixing and supplying the necessary oxygen to satisfy the organic loads imposed on the lagoon. A discussion of the combined results is presented in section 7.5.

7.2 Evaluation of Lagoon Mixing/Flow Characteristics with Various Aeration Schemes.

The lagoon was originally designed to contain nine (9) equally-spaced aerators. The current configuration is as shown on Figure 13, with eight irregularly spaced units. Random down time of the units resulted in numerous combinations of aerators in operation during the lagoon evaluation period. Lagoon flow and mixing patterns could not be directly assessed for all such combinations. Several studies were performed to ascertain the effects of aeration on lagoon flow patterns. Documentation of short-circuiting and limited detention times was needed for data reduction and analysis. Preliminary evidence of severe short-circuiting without aerator operation was determined early in the study when dye discharged into the influent and surface visual observations were made. The apparent surface flow pattern is shown in Figure 13. The lagoon influent rate at this time was 3 mgd.

Several aeration schemes were evaluated using dye tracer studies including "no aerators on", "all aerators on", and a combination of "aerators off and on." This latter combination was typical of the actual operation that occurred for a considerable period of lagoon evaluation. In addition to entire lagoon mixing

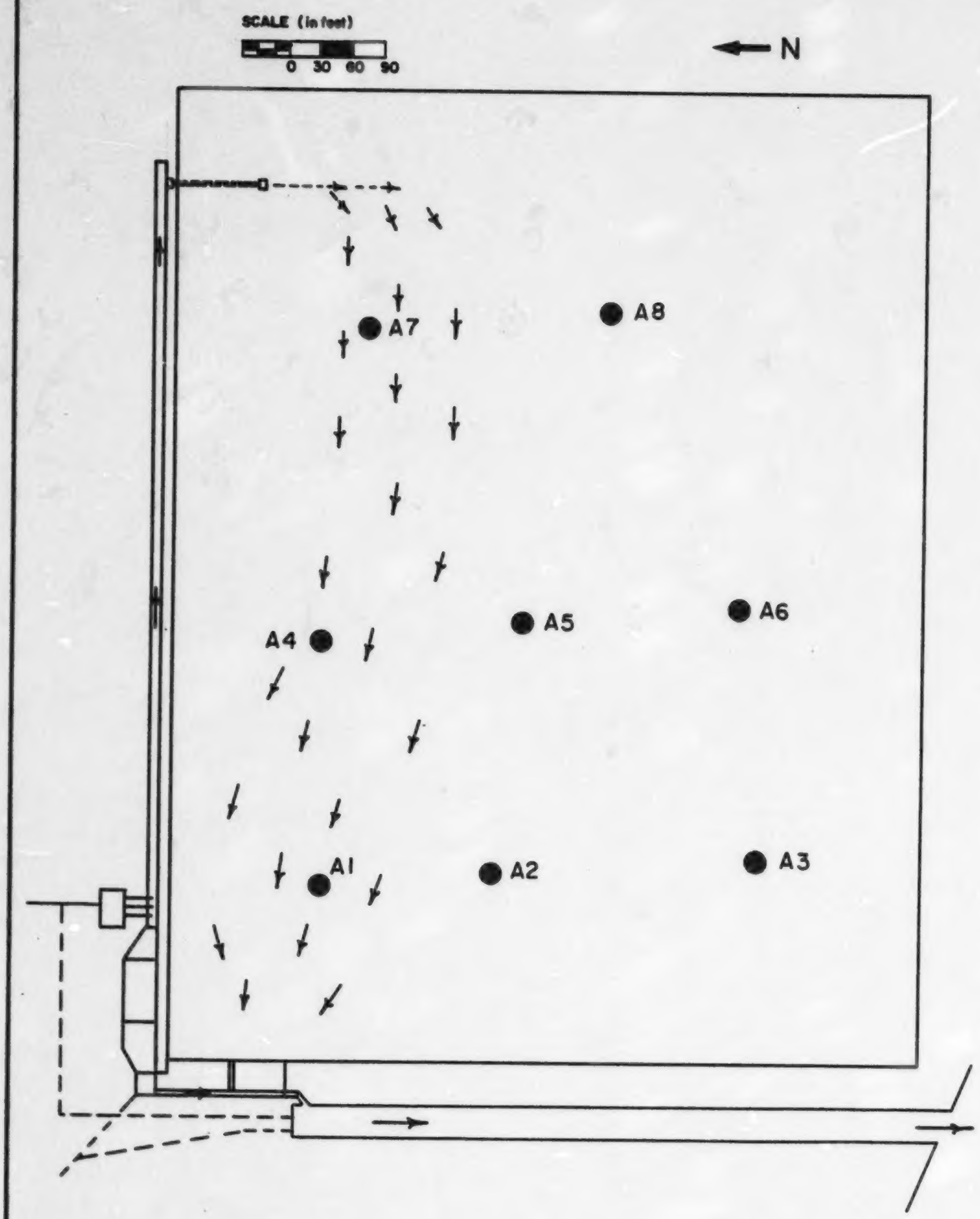

SCALE (in feet)
0 30 60 90
N
A1
A2
A3
A4
A5
A6
A7
A8

Figure 13. Preliminary dye tracer study (3 mgd discharge).

evaluation, an analysis of the flow characteristics in the vicinity of a single aerator was conducted by means of flow direction and velocity measurements.

#1 Dye Study

Since none of the aerators were operational for much of the lagoon evaluation period the first dye tracer study was performed without aerators in operation at a flow rate of 18 mgd of 32 sampling points with additional samples taken at the effluent channel. Without aerator mixing, it was suspected that short circuiting would create a pattern of direct flow from the northeast corner at the influent pipe to the northwest corner where the effluent is located.

Rhodamine B was added to the lagoon in sufficient quantity to yield .01 mg/1 of dye concentration throughout the lagoon (assuming complete mixing). Samples were taken at one foot and 15 feet depths and analyzed for dye concentrations. Sampling enphasis was placed on numerous sampling locations and only 2 sampling depths since short circuitry was expected to cause minimal detention time within the lagoon thus precluding detailed depth sampling. Resulting flow patterns are shown in Figure 14 for the 1-foot depth. The locations of the 32 dye sampling stations are also indicated in Figure 14.

A comparison of the patterns shown in Figures 13 and 14 indicate that the additional discharge results in a further southerly displacement of the influent waste stream across the lagoon. The influent movement is predominantly surface-oriented. Lagoon discharge is by surface overflow at the northwest corner, causing a drawdown toward this point throughout the water body.

Flow patterns within the lagoon without aeration are then subject to these two forces besides variable wind effects. The condition of"no aerators on" results in little lagoon mixing with significant surface flows. Mixing was nearly complete within the lagoon surface water as shown by the high concentrations throughout the area at the end of the study. Mixing with lagoon subsurface and bottom waters was minimal as was observed by the dye concentrations at the 15-foot depth.

Lagoon detention time can be estimated by inspection of the effluent dye concentrations shown in Figure 15. The results indicate that the peak dye effluent concentration occurred 3 hours after dye injection. Residual dye was recorded in the effluent for a total of 30 hours. On a mass basis, roughly 50% of all the dye recorded in the effluent had passed after six hours. Given that the flow rate was 18 mgd the 129 million gallon volume within the lagoon had a potential detention time of 7 days. The lack of mixing without aeration clearly reduces the actual detention time.

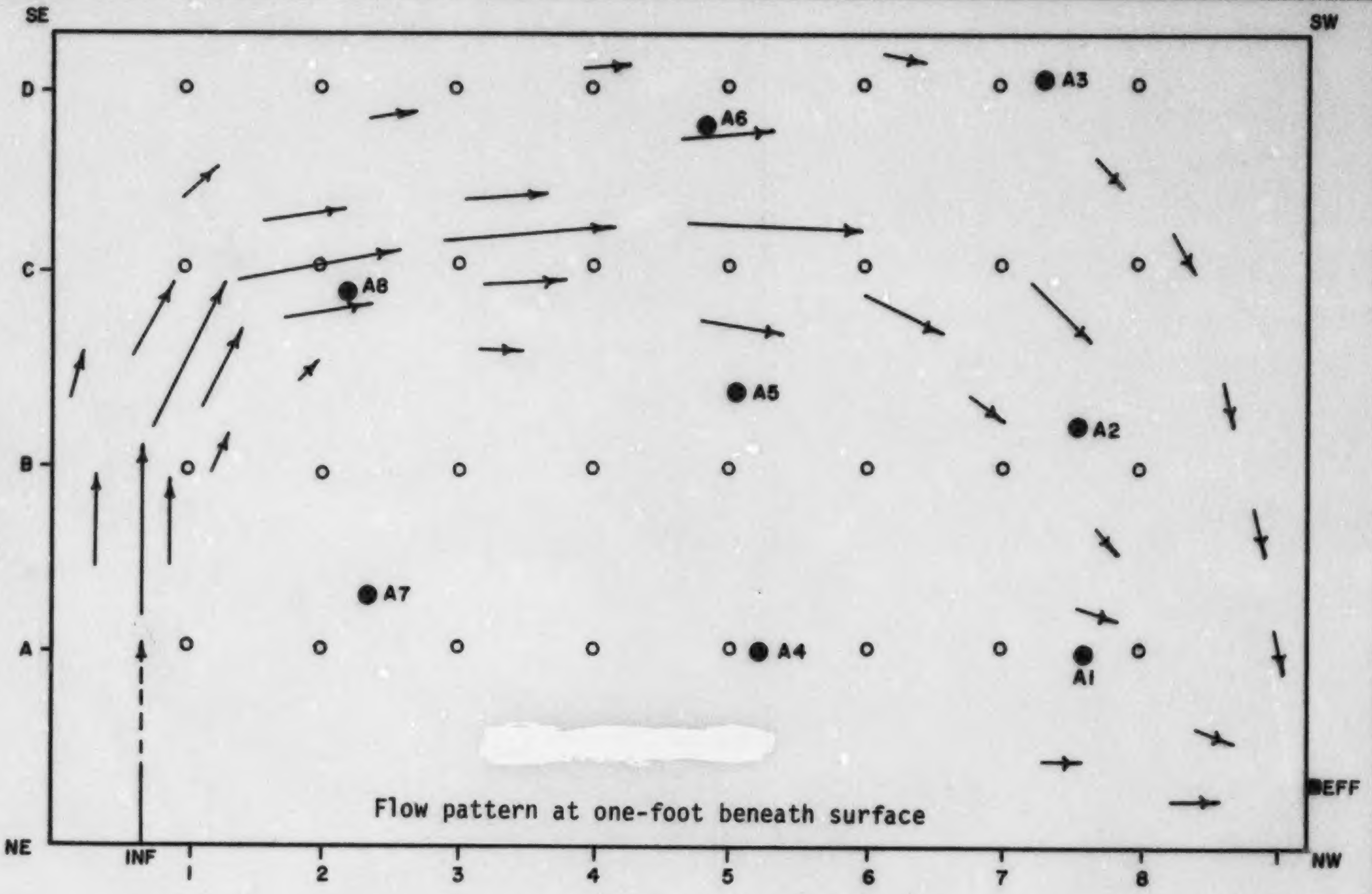

SE
SW
NE
NW
D
C
B
A
INF
EFF
1
2
3
4
5
6
7
8
A1
A2
A3
A4
A5
A6
A7
A8
Flow pattern at one-foot beneath surface

Figure 14. Results of dye tracer study (18 mgd discharge).

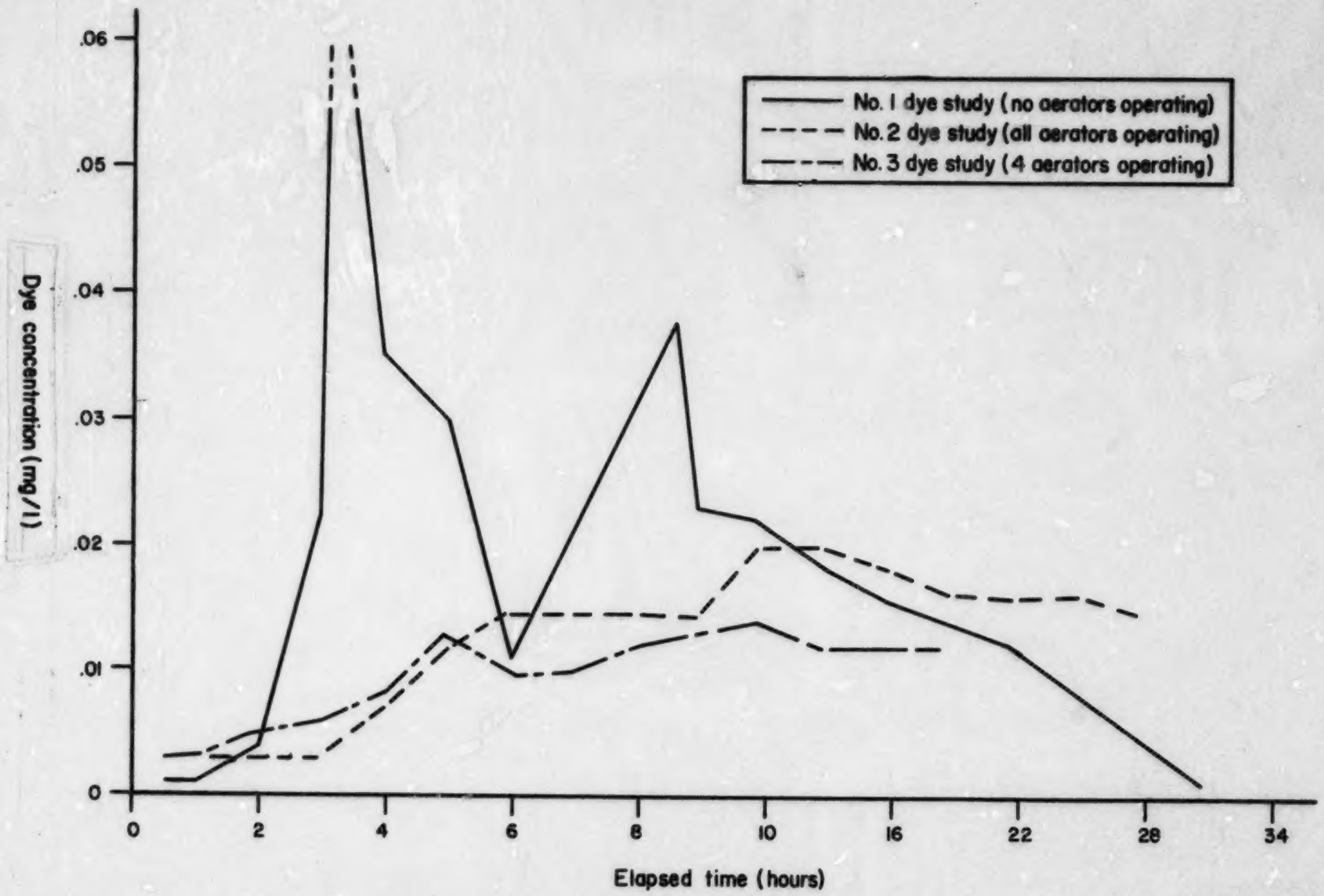


Figure 15. Lagoon effluent dye tracer concentrations versus time.

#2 Dye Study

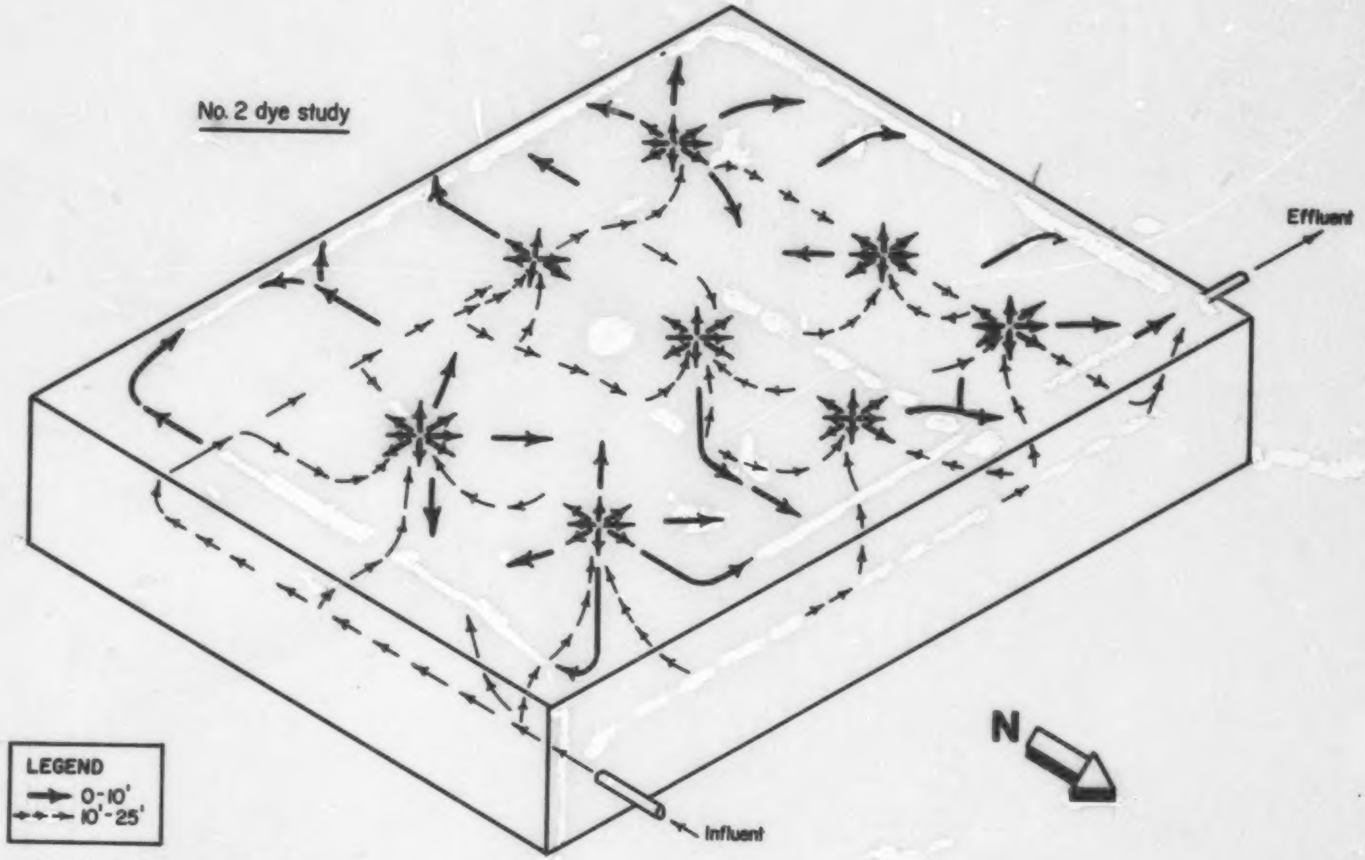
A second dye study was performed with the lagoon inflow at 18 mgd and with all eight aerators in operation. In order to more clearly define the extent of mixing depth samples were collected at 1-foot, 10-foot and 25-foot intervals. Fewer stations were sampled due to the increased time required to sample the additional depths and the need to adequately track the dye dispersion.

Flow patterns during this study initially indicated the characteristic flow across the east side. After 30 minutes the flow migrated toward the effluent channel but the degree of vertical mixing was significant. Unlike the evaluation without aeration, no extremely high dye concentrations occurred. Areas of higher than average concentration in this study were always quite large, with actual concentrations being only several times the expected complete mix values. An isometric view of the observed profiles is depicted in Figure 16. After about 3 hours, the dye concentration throughout the lagoon was nearly equal, demonstrating complete mixing of the system at all depths. Six hours after the initial dye injection, the equivalence of lagoon dye concentration still indicated a completely mixed system. Final dye concentration evaluation was made throughout the lagoon after 24 hours. As expected, all residual dye concentrations were equivalent and showed minimal reduction over the elapsed time period.

Evaluation of effluent dye concentrations produced a pattern of increasing concentration for the first six hours and then for the next 30 hours, at which time the evaluation was ended. It can be concluded that the operation of all aerators completely mixes the lagoon waters. At the given flow rate of 18 mgd, the theoretical detention time of 7 days could not be completely evaluated by dye tracer methods due to the length of this time period. The verification of the system as being completely mixed justifies the interpretation of the actual detention time as being the theoretical detention time.

#3 Dye Study

Aerator operational difficulties created the situation in which there typically were one or more aerators not functioning during the lagoon evaluation period. Each aerator malfunction created a change in lagoon flow pattern and ultimately the overall detention time. During the period of August through September, 1979, extensive data was collected, while many of the aerators were non-operational. At most times, only four aerators 1, 3, 4 and 6 were functioning (see Figure 13).


No. 2 dye study
Effluent
Influent
N
LEGEND
0-10'
10'-25'

Note: Vertical exaggeration = 2 X

Figure 16. Isometric view of lagoon flow pattern, all aerators in operation.

A dye tracer study was performed using this aeration scheme to typify semi-aeration mixing performance and to aid evaluation of other data collected under this condition.

In the two prior studies dye concentration and movement indicated that the initial discharge velocity was sufficient to create currents flowing almost completely across the lagoon. The aeration pattern during this evaluation differed in that the aerators disrupted the effluent current yet were not proximate to the influent to create mixing upon initial discharge. The resulting flow pattern allowed the water discharged at the lagoon bottom to flow directly across toward the far side with minimal mixing. Aerators 4 and 6 then drew bottom water toward the units. During this period, low dye concentrations were found in upper water. Once the dye reached these aerator units, it was dispersed in all directions within the surface waters. This created a surface flow back toward the influent. The net result is rapid mixing in the vicinity of the aerators and a slower, but still complete, mixing in the unaerated east end of the lagoon. Dye laden waters propelled toward the effluent were subjected to further mixing at aerators 1 and 2. The effects are similar to the system with all aerators operating in which complete mixing is provided. The southwest corner of the lagoon is some distance from the nearest active unit. Mixing of this area lags behind the rest of the lagoon but eventually, complete mixing occurrs. Review of effluent dye concentrations plot shown in Figure 15 shows a steady rise to a plateau after 5 hours which remains steady for an additional 20 hours at which time sampling was ceased. This flow pattern is almost identical to that of the complete aeration system with the exception of the time required to reach the plateau. The decrease by an hour of the time to peak may be due to the swiftness of flow along the bottom to the center aerators, whereas under the full aeration scheme, the east- end aerators acted as a barrier to direct flow. With this aeration scheme, the actual overall detention time is somewhat less than the theoretical based on volume and flow rate.

7.3 Evaluation of a Single Aerator Mixing Characteristics

The aerators draw bottom waters toward the units, while dispersing surface waters radially outward. Analyses of flow patterns in the vicinity of the number 4 aerator were performed using the Marsh McBinney electromagnetic flow meter. Measurements were taken at several distances from the aerator and at several depths. In general, the aerator causes surface water down to a depth of 5 to 10 feet to flow away from the unit while drawing waters below this depth slowly inward and upward. The direction and velocity of the aerator-induced flow at several distances is depicted in Figure 17. As expected, flow velocities decrease with distance from the units.

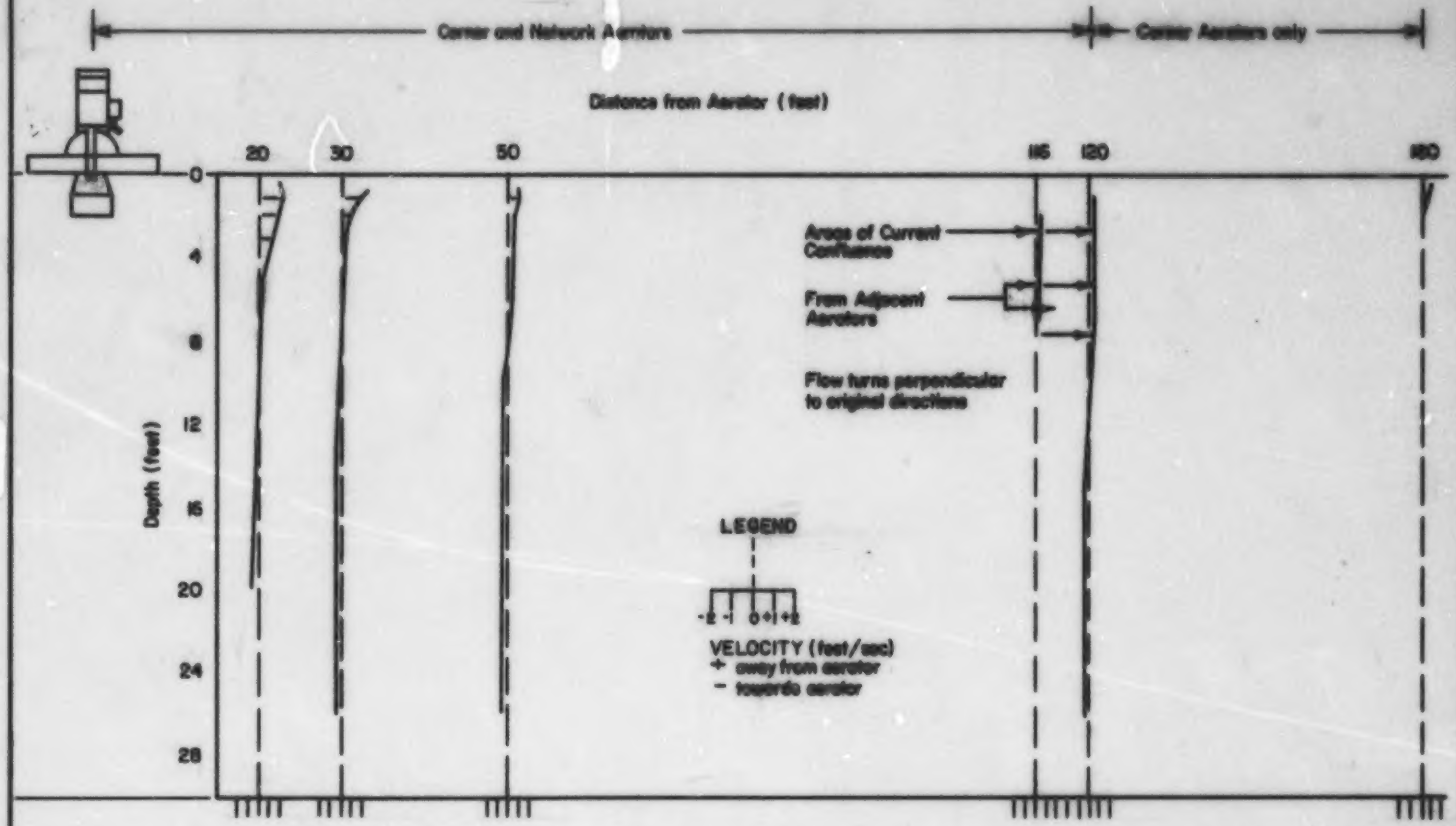


Figure 17. Typical aerator induced flow patterns.

In the area where surface flows from aerators 1 and 4 interacted, additional velocity measurements were performed. It was determined that approximately 120 feet from each aerator surface flows turn perpendicular to each oncoming flow in a direction away from shore. Water below the 10-foot depth continued to flow toward the aerators. Additional velocity measurements were taken in the lagoon corners at distances up to 160 feet from the nearest aerator. Results yielded surface velocities of 0.2 to 0.7 fps (6.1 - 21.2 cm/s) under conditions without wind or other apparent extraneous effects.

Based on the present locations of the aerators, it appears that complete lagoon mixing would be expected with all units operating.

The results of this experiment along with the tracer dye studies indicate that four aerators are adequate to mix the lagoon. The actual flow pattern appears to yield various slug flow features. Part of the initial flow is fed back toward the influent, part is completely mixed and reaches the effluent, and part is forced into remote areas of slow turnover.

7.4 Lagoon Dissolved Oxygen Concentration Patterns

One of the more significant parameters for determination of operating conditions is the dissolved oxygen concentrations within the lagoon. During the evaluation period, measurements were made every other day at the lagoon influent, effluent and 12 interlagoon sampling points. In addition, complete lagoon profiles were performed measuring the dissolved oxygen concentrations and temperatures at 32 locations througout the lagoon evaluated at four depths.

Observations February-May, 1979

In late February of 1979, lagoon operations began at an influent flow of 3 mgd after an extensive period of no flow. Inflow remained at 3 mgd for two months during which lagoon oxygen levels remained above 10 mg/l. Water temperatures increased from near freezing to about 10 degrees centigrade. Limited mechanical aeration was provided during this period with the exception of a two week period in early April. Lack of aeration did not result in any decrease in dissolved oxygen levels.

Several days after aeration resumed lagoon inflow ceased for a 2-1/2 week period during rehabilitation of the additional secondary lift pumps. Aerators which had operated intermittently were again all non-functional. Oxygen levels within the lagoon fell to an average of 6 mg/l during this time. Restart of inflow was limited to 3 mgd for the first two weeks in which time no mechanical aeration was provided in the basin. Oxygen levels were above 5 mg/l in all but the very bottom waters. Over a period of several days, dissolved oxygen levels began decreasing slowly. However, before a definite trend was developed, the rehabilitated secondary lift pumps became operational and on May 22, 1979, lagoon flow increased to 18 mgd.

Dissolved Oxygen Lagoon Profile - May 26, 1979

Dissolved oxygen levels began to fall sharply and on May 26, 1979 a complete lagoon profile was undertaken. Dissolved oxygen levels were all less than 2.5 mg/l, with the general trend being a slight concentration increase moving from the influent to the effluent end of the lagoon. Variations in dissolved oxygen levels with depth were minimal to 25 feet with very low oxygen concentrations in bottom waters. Dissolved oxygen levels continued to fall throughout the lagoon, with low concentration remaining for the next few weeks.

The results of the profile indicated that during periods of high flow and no aeration, lagoon waters are nearly homogeneous with respect to dissolved oxygen and temperature. Only the center area of the north side varies from the general trend. In this area oxygen levels were slightly higher indicating less organic loading which is consistent with the dye studies performed reported in this chapter. Each of these studies demonstrated that the influent water tends to flow to the far side of the lagoon and travel along the south side unless interrupted by aeration.

In addition to the increased inflow volume, the lagoon operations were subjected to increased organic loading on a concentration basis. For a few days, prior to the operation of the additional lift pumps, concentrations of most of the pollution parameters had risen to three times previous values. This additional organic demand effectively caused the slow decrease in lagoon oxygen levels. Due to the large number of constituents which increased in concentration, it is likely that the secondary treatment plant was undergoing operation problems. Increased concentrations of heavy metals entering the lagoon indicates this possibility, as chromium, lead and zinc levels increased threefold at this same time.

Dissolved Oxygen Profile, June 6, 1979

For two weeks after the May 25 dissolved oxygen profile, all of the influent pumps were in operation supplying a total of 18 mgd to the lagoon. None of the aerators were operational which was reflected in the low oxygen levels. The trend of decreasing oxygen levels which began prior to May 25 continued until May 31. Representative daily samples indicated dissolved oxygen concentrations had fallen to 1.0 mg/l by May 26 and to less than 0.5 mg/l by May 31. On June 1, 0.3 inches of rain occurred with an additional 0.1 inch falling on June 5. On June 6, a second dissolved oxygen profile was performed to evaluate the extent of the low oxygen levels. Although these storms occurred prior to the June 6 profile, oxygen levels did not change at the daily sampling sites from prior to the storms until several days after the profile. Results of the June 6 profile proved these low levels to be representative of the entire lagoon, oxygen levels dissolved less than 0.7 mg/l at all locations. Oxygen concentrations were higher at the effluent end of the lagoon, lower along the influent end, and lowest in the central portions. Dissolved oxygen levels were always greater than 4 mg/l in the influent channel, which raises the oxygen levels in lagoon areas of initial contact. The significant feature of this profile was the total agreement of the selected locations regularly tested every other day to the overall lagoon profile.

Continued low oxygen levels were a function of several operational features. First, temperatures had been increasing steadily since early March, with a corresponding decrease in saturation point. Lagoon waters dissolved oxygen values were so low that temperature effects on reaeration constants were insignificant, but at this increased temperature influent dissolved oxygen levels were also limited. The influent channel is characterized by relatively turbulent flow providing good opportunity for reaeration throughout its length, especially at the pump discharges and the outlet overflow point. Increased temperatures significantly decreased the extent of influent reaeration, thus limiting this oxygen supply to the lagoon. Second, the concentration of constituents entering the lagoon had increased sharply during the week prior to the May 25 profile. Concentrations decreased to typical levels after this time. However, lagoon pollutional loading had become sufficient to begin the trend of oxygen depletion.

The third cause of continued oxygen depletion was the increased flow rate from 3 to 18 mgd. Although influent pollutant concentrations decreased, overall lagoon loadings increased. Observations of inter-lagoon oxygen levels over the following weeks indicated no improvement over the low levels recorded during the June 6 profile.

Dissolved Oxygen Profiles: June 30-July 4, 1979

During the latter part of June, 1979, the influent flow had been at 18 mgd for four weeks and dissolved oxygen concentrations remained less than 1 mg/1. On June 26, three aerators were started and by June 27, seven aerators were in operation. The increase in dissolved oxygen was immediate and continued rising such that on June 30, a program was initiated to survey the lagoon oxygen levels on several consecutive days.

The first profile revealed high dissolved oxygen levels (6.0 mg/l) in all except the very bottom waters. Variation throughout the lagoon was minimal, both lengthwise and with changes in depth. The slight variation present was limited to the south side central area where levels were about 0.5 mg/l, lower than found elsewhere. Oxygen levels found in the southeast corner implied no adverse affects due to lack of operation of the one nearby aerator.

In general, it appeared that all areas were evenly mixed with higher organic loading in the south central area, causing slightly reduced dissolved oxygen values. This trend is plausible in light of the known tendency of influent waters to flow to the far side of the lagoon during high flow conditions. It appears that aeration effectively, though not completely, broke up this pattern, redistributing pollutional loading through the lagoon.

A second profile performed on July 1 indicated dissolved oxygen increases of between .1 and 1.0 mg/l. The greatest increase occurred in the areas lowest in dissolved oxygen on June 30, effectively decreasing the overall lagoon variation. The same general trends were still maintained. Additional monitoring points were selected within the basin to further define the depth of high oxygen levels since little variation had been seen down to depths of 25 feet. Analysis at point 2 feet from the bottom indicated minimal change from concentrations seen in the main lagoon body. This indicates that the oxygen gradient from high dissolved oxygen levels to less than 0.5 mg/l occurrs very close to the benthic layer and implies a reasonable exchange rate of bottom waters.

The third consecutive profile, performed on July 2, indicated an additional increase in oxygen levels in those areas which had been lowest on July 1. Areas of highest oxygen concentration, 7.0 mg/l, remained at that level. Following the same trend as the day before, the changes in oxygen concentrations were slight and effectively decreased the overall variation within the lagoon. Dissolved oxygen levels were between 6.5 and 7.0 mg/l in all but benthic areas and were at 80% of the saturation level.

The lagoon which had contained minimal oxygen concentrations was effectively restored to 70 percent saturation after five days of aeration and stabilized at 80 % of saturation after a total of seven days. A review of the data compiled at the regularly monitored locations indicated that nearly 60 percent of saturation was accomplished during the first day of aeration. Influent BOD levels averaged 30 mg/l during this period whereas the 3 month average was 22 mg/l, thus demonstrating the capability of aerators to increase lagoon oxygen levels during a period of relatively high organic loading. On the basis of lagoon reoxygenation, there appears to be no need for continual operation of all aerators within the lagoon. Since seven of the available eight were capable of restoring an oxygen-depleted lagoon to 80 percent saturation, normal operating conditions would require less reaeration to maintain high oxygen concentrations.

Lagoon Oxygen Trends July 4 - August 18, 1979

For a six-week period after July 4, average dissolved oxygen levels within the lagoon ranged between 6 to 8 mg/l with five to seven aerators operating. On August 14, the average dissolved oxygen level within the lagoon was recorded at 4.7 mg/l. Several operational characteristics combined to produce this effect.

Beginning on August 9, a three day storm period occurred with a total of 1.8 inches of rain. Combined sewer overflow into the lagoon was not recorded, but based on other storm periods, it was estimated to be several million gallons. Influent BOD concentrations which had averaged 13 mg/l for several weeks were 48 mg/l on August 13. TSS, TOC, NO_3 and TP values underwent similar increases. For several days prior to August 14 other evaluations precluded the usual sampling schedule such that little lagoon loading data is available. Average interlagoon BOD values had increased from 16 mg/l on August 8 to 32 mg/l on August 14. This increase could not have occurred solely due to the high inflow concentration of the 13th, implying high organic dry weather loadings for much of the period between the 8th and the 13th. Based on the early July analysis the lagoon aeration system is normally capable of reaeration rates satisfying the organic demand. During this specific instance, however, aerator failures left only four in operation. Neither of the two aerators positioned to initially mix influent flows were operational. By August 18th, lagoon oxygen levels had increased to above 8 mg/l although no additional aerators became operational.

This time period depicts the typical response of the lagoon to high influent loading periods and indicates that under limited aeration an oxygen deficit would occur. Similar conditions existed in late August and a more extensive data set was collected for a more complete evaluation of lagoon response.

Dissolved Oxygen Profile, August 30, 1979

In the middle of August, 1979, the influent flow rate was 15 mgd and four aerators were in operation. Dissolved oxygen levels throughout the lagoon varied with time ranging from 6 mg/l to 9 mg/l over the course of several weeks. During the fourth week of August, two storm events occurred and the dissolved oxygen levels diminished sharply. On August 23, a rain storm occurred resulting in 1.4 million gallons of CSO entering the lagoon. The storm water inflow contained concentrations of most constituents similar to that of the secondary treatment plant effluent normally entering the lagoon. The effect of the storm was a temporary increase in inflow rate with little concentration change of parameters throughout the lagoon. Oxygen levels throughout the lagoon averaged 9 mg/l on August 22. These levels dropped to less than 6 mg/l by August 24 and remained at this level for several days.

A second storm occurred on August 27 discharging 21 million gallons of CSO into the lagoon. Concentrations were significantly higher than normal influent levels for all parameters except nitrate and total solids. Sufficient quantities of these pollutants were discharged in the lagoon to increase actual interlagoon concentrations by 10 to 50 percent. No additional oxygen depletion was observed below the August 24 levels.

On August 30, a complete lagoon dissolved oxygen profile was performed. During the week prior to this study, normal lagoon influent flows also had high BOD levels, probably due to the increased loading effects of the storms on the treatment plant. The combined result of these inputs was an increase of BOD values throughout the lagoon from less than 10 mg/l to about 40 mg/l. TSS and TOC followed a similar trend, whereas NO_3 and TP levels remained steady. A dissolved oxygen profile was performed with a number of aerators in operation to determine the effect of partial aeration during this high organic demand period. Aerators 1,2 4 and 6 were in operation. The relative positions of these units implies little initial aeration of the influent flow with good aeration along the sides at the center of the lagoon and at the effluent end. The dissolved oxygen profile indicated the expected trend in which decreasing oxygen levels occurred with increased distance from the operating aerators. Lowest dissolved oxygen levels are found along the east end and in the south-west corner where no aerators were operating nearby.

Although the aerator in the center of the lagoon was not operating, oxygen levels were not as low as at other non-aerated locations. It appears that central lagoon waters are sufficiently influenced by currents created by distant aerators to cause rapid changeover as opposed to remote, non-aerated areas, of low exchange rate.

Since oxygen levels actually increased from the influent to the effluent-end of the lagoon, it appears that four aerators were sufficient to meet operating requirements of the lagoon under the conditions present. Consideration must then be given to the mixing characteristics of the aerators in combination with inflow and outflow induced currents to ensure complete utilization of lagoon volume to maximize treatment time.

General Observations

It was determined during all of the dissolved oxygen profiles performed in the lagoon that minimal decrease in concentration occurs with increasing depth down to and including 25 feet. The lagoon bottom typically ranges from 28 to 32 feet and results indicate that waters within one foot of bottom sediments consistently contain less than 1 mg/l of dissolved oxygen. It appears that most of the concentration gradient occurs within the first few feet as spot measurements taken two feet from the bottom indicate concentrations consistent with those found in upper waters.

One bottom area consistently indicated high DO concentrations. This location at the point of influent discharge is constantly subject to high horizontal flow rates imparting scouring characteristics. This flow pattern effectively yields high oxygen concentrations in two ways. First, limitation of solids buildup limits the benthic oxygen demand. Second, high flow velocities create continued water exchange with the high dissolved oxygen of the influent.

CHAPTER 8

LAGOON TREATMENT RATES

8.1 Foreword

Statistical summaries of the lagoon pollutant removal performance are presented in this chapter. Overall lagoon performance evaluation results for three different influent discharge regimes are presented in section 8.2. A comparative analysis of lagoon removal efficiences with and without aeration is given in section 8.3. An analysis of heavy metal characteristics is presented in section 8.4. Estimates of lagoon performance under wet weather CSO treatment conditions are given in section 8.5.

8.2 Lagoon Performance Evaluation/Different Discharge Rates

Pollutant loadings into the lagoon were a function of the ECSD WWTP flow rate and the concentrations of individual constituents within the treatment plant effluent. Influent flow to the lagoon was controlled on a volume basis by the operation of the lift pumps. Concentrations of waste water constituents were mainly a function of the degree treatment provided by the secondary plant. Due to combined effects of erratic influent pollutional loads and variable treatment efficiency the treatment plant effluent concentrations varied widely.

The evaluation period was divided into several phases based on prevalent influent rates to the lagoon. Two of these periods were purposely selected for direct evaluation and comparison, whereas the remaining periods of extended no flow and minimal flow conditions were simply due to operational problems. The total duration of evaluation lasted for 56 weeks. Six weeks were without any inflow. The influent rate was 3 mgd for 33 weeks. Fifteen weeks of evaluation were conducted at an influent rate of 18 mgd which is the maximum inflow rate provided by all three pumps. The final two weeks of the evaluation program were conducted at a reduced flow rate of 11.3 mgd. Flows were reduced during this period to aid the evaluation of detention time on lagoon performance and to limit the amount of river water pumped into the lagoon through bypass pipes during low flow portion of the diurnal cycle. Analyses of different data groupings in this section do not reflect differences in aerator operations. These differences are reported in the section 8.3.

Statistical reduction of the lagoon performance data are shown in Table 3 for the 3 evaluation periods. The mean, number of samples, standard deviation and coefficient of variation (expressed as a percentage) are provided in Table 3 for the influent and effluent pollutant concentrations. With the exception of the organic pollutants the influent concentrations were reasonably consistent over the three evaluation periods. The organic contaniments increases during the low flow period could have been due to seasonal changes since the cold weather may have reduced treatment plant effectiveness.

For ease in comparison, the average pollutant removals for the three flow rates are summarized in Table 4. Effluent concentrations are listed as averages of the flow period. Percentage removals for each flow rate were determined using the average influent and effluent concentrations. The last column of the right hand side of Table 4 shows the overall removal efficiency considering bypassed flows under the pumpage rate of 11.3 mgd. No flows are bypassed at a discharge rate of 18.8 mgd.

The results shown in Table 4 are expected considering the continued nature of difficulties with the lagoon's operation. Little aeration was provided during most of the 3 mgd flow period. Although long detention periods were theoretically possible during this period, severe short-circuiting of influent flow effectively minimized the actual treatment time. Lagoon removal efficiencies for the 11.3 mgd flow rate were higher than those computed at the higher rate of 18.8 mgd. This result is expected as a theoretical detention time of 12 days, is possible at 11.3 mgd while only seven days of detention are provided at 18 mgd. Aeration/dye tracer studies reported in chapter 7 have shown that even with the limited aeration provided at these flow rates, nearly complete lagoon mixing occurred. Influent pollutant concentrations were quite similar and temperature variations were minimal, thereby eliminating the possible effects of these variables.

8.3 Comparative Analysis of Lagoon Removal Efficiences With and Without Aeration

In Chapter 7 the analysis of flow patterns within the lagoon under differing aeration schemes indicated large variations in the apparent degree of mixing. During the high-flow rate period of evaluation (18.8 mgd) extended times of aeration and of non-aeration occurred.

Several days after the startup of the rehabilitated lift pumps, oxygen levels stabilized at low levels. The high flow rate/no aeration period began at this time and was characterized by the oxygen levels below 0.5 mg/l and temperatures near 20 degrees centigrade. Removal efficiences for this period are listed in Table 5. Average influent concentration of most

TABLE 3

STATISTICAL SUMMARIES OF POLLUTANT REMOVAL EFFECTIVENESS FOR THREE INFLUENT FLOW RATES (3., 11.3 and 18.8 mgd)

A. Influent Rate = 3 mgd (August, 1978 - April, 1979)

	Influent				Effluent			
Para-Meter	Mean Conc.	No. Samples	Std. Devn.	CV%	Mean Conc.	No. Samples	Std. Devn.	CV%
BODe	36	1	-	-	17	69	13.9	82
BOD_5	44	28	33.4	76	30	28	29.5	105
COD	155	60	49.5	32	83	60	30.1	36
TOC	47	81	45	96	28	81	10.9	39
TSS	168	71	267	159	35	71	23	66
VSS	75	68	110	147	24	68	20	83
TS	1585	52	757	48	1365	62	579	50
NO_3	-	-	-	-	-	-	-	-
TP	1.30	52	1.49	115	.57	62	.80	140
TURB	80	78	51	64	40	78	10	25

B. Influent Rate = 11.3 mgd (August - September, 1979)

	Influent				Effluent			
Para-meter	Mean Conc.	No. Samp.	Std. Dev.	Coeff. Var.	Mean Conc.	No. Samp.	Std. Dev.	Coeff. Var.
BODe	21	12	11	92	22	12	8.5	37
BOD_5	23	12	17.3	75	13	12	8.3	64
COD	117	14	112	96	57	14	19	33
TOC	39	12	10.7	27	26	12	5.7	22
TSS	230	15	407	177	16	15	6.6	41
VSS	68	15	79	116	9	15	5.3	59
TS	1639	15	370	23	1323	15	201.0	15
NO_3	6.3	15	3.7	59	6.1	15	4.7	77
TP	1.13	14	.67	59	.23	14	.12	52
TURB	113	13	61	54	21	13	10.7	51

C. Influent Rate = 18.8 mgd (May - August, 1979)

	Influent				Effluent			
Para-meter	Mean Conc.	No. Samp.	Std. Dev.	Coeff. Var.	Mean Conc.	No. Samp.	Std. Dev.	Coeff. Var.
BOD_e	18	17	8.8	49	26	19	18.5	71
BOD_5	22	17	10.3	47	21	19	18.5	88
COD	120	1	-	-	79	23	34.7	44
TOC	40	23	17.0	42	28	23	9.6	34
TSS	144	25	147	102	32	25	11.4	36
VSS	60	25	51	85	21	25	9.0	43
TS	1626	25	333	20	1571	1	-	-
NO_3	5.5	1	-	-	5.5	1	-	-
TP	1.24	23	.72	58	.34	23	.22	65
TURB	83	22	54	65	21	23	9.6	42

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD; BEGIN SECTIONS HERE

TABLE 4

POLLUTANT SUMMARY OF AVERAGE REMOVAL EFFECTIVENESS FOR THREE INFLUENT FLOW RATES (3.0, 11.3 and 18.8 mgd)

	RATE = 3 mgd			RATE = 11.3 mgd			RATE = 18.8 mgd			
Parameters	Influent*	Effluent*	Removal (%)	Influent*	Effluent*	Removal (%)	Influent*	Effluent*	Removal (%)	Removal(%) Overall at 11.3 mgd
BOD_e	36	17	53	21	22	-	18	26	-	-
BOD_5	44	30	32	23.0	12.7	45	22	21	4	27
COD	155	83	46	117	57	51	120	79	34	31
TOC	47	28	42	38.6	25.9	33	40	28	31	20
TSS	168	35	79	230	16	93	144	32	78	56
VSS	75	24	68	68	9	86	60	21	65	52
TS	1585	1365	14	1639	1323	19	1626	1571	3	11
NO_3	-	-	-	6.3	6.1	3	5.5	5.5	0	2
TP	1.3	.57	56	1.13	.23	80	1.24	.34	73	48
TURB	80	40	49	113	21	91	83	21	75	55

* Mean Concentrations (mg/l)

** Overall Removal at 11.3 mgd rate considering untreated bypassed flows.

BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS

EPA-287 (Cin.)

constituents were similar during periods of aeration and no aeration. Analyses of aeration effects were made by selecting a period of time after sufficient aeration occurred to replenish the dissolved oxygen levels in the lagoon. Removal efficiencies under the high oxygen, complete mix conditions are also listed in Table 5. Comparison of the removal rates indicate that each of these treatment schemes provided better removal for certain parameters.

As had been seen throughout this study the electrolyte BOD analysis failed to show any reduction of this parameter in the lagoon. BOD and COD show a slightly better removal rate with aeration. All other parameters tend to receive better treatment without the aerators operating. Without aeration, improved nitrate removal is the result of denitrification within zones of depleted oxygen. Oxygen levels within the lagoon were reduced to less than 0.5 mg/l yet the rate of removal of other constituents did not diminish. With aeration, induced currents limit the settling of solids as reflected in the decreased removals.

It appears that the disadvantage of reduced detention time is more than compensated by the lesser degree of flow turbulence. The removal trends seen here are then explainable as follows. Parameters which require lengthy reaction times and reasonable oxygen levels, such as BOD and COD, receive a greater degree of treatment under conditions conducive to complete mixing and reoxygenation, that is adequate aeration operation. Suspended solids and the large mass of fine solids require quiescent conditions for optimal removal, as provided in the lagoon without aeration. Those parameters which are typically tied up in the solids particulates such as TP, indicate similar removal rates and are therefore primarily removed by settling. Reductions in parameters such as BOD and COD which are impeded by decreased oxygen levels, are also related to the solids fraction and thus undergo removals by this mechanism. The data indicates that this mechanism is less effective. It can be surmised that optimum treatment conditions would imply a water body of high oxygen content and be sufficiently quiescent to promote particulate settling. In the case of a treatment lagoon this could be accomplished using initial aeration to levels beyond the depletion capability of the wastewater followed by a zone of quiescent flow toward the effluent maximizing settling conditions.

The higher inflow rate of 18.8 mgd is nevertheless deemed optimum. This conclusion is based on the overall removal rate including bypassed discharges. At the 11.3 mgd pumping rate, excess secondary treatment plant effluent is discharged to the river without further treatment. For the purposes of this consideration assume that the occurrences of reversed river water pumping can be ignored so that all of the flow at 18.8 mgd is secondary plant effluent. Comparing the actual mass of pollutant

TABLE 5

Summary of Mean Pollutant Removal Rates Under Different Aeration Schemes

(Flow Rate = 18.8 mgd)

	Parameter	Influent Mean	Effluent Mean	Removal(%)
A.	BOD_e	21	34	-
	BOD_5	25	23	6
	COD	139	91	35
	TOC	40	27	32
	TSS	192	32	83
	VSS	76	23	69
	TS	1782	1607	10
	NO_3	2.21	.94	57
	TP	1.42	.26	82
	TURB	97	17	83
B.	BOD_e	16	17	-
	BOD_5	20	18	11
	COD	93	58	37
	TOC	41	31	24
	TSS	89	34	62
	VSS	42	19	54
	TS	-	-	-
	NO_3	3.40	3.54	-
	TP	0.92	0.33	63
	TURB	-	-	-

A. No Aeration (5/26 - 6/25, 1979)

B. With Aeration (7/7-8/7, 1979)

reduction of the treated 11.3 mgd to the total loading yields the removal percentages as indicated in Table 4. The removal rates at the 11.3 mgd flow rate are less than those representing lagoon treatment of the entire waste flow, with the exceptions of BOD, TS and NO_3. NO_3 removals are low in both situations. Total solids and BOD removals were imparted by four storm events during the higher flow period. It will be discussed in Section 8.4 that these occurrences caused pollutant washout before adequate treatment could be provided and resulted in high lagoon residual levels.

Limited removal of total solids during both flow evaluation periods is a result of the poor, small fine particle settling within the lagoon. Although suspended solids removals were high, these loadings represent only a small fraction of the total solids load. The results suggest that the settling velocities of the fine solid particulates are insufficient to overcome lagoon currents and scouring near aerators. This problem was further complicated during the evaluation by the high occurrence of aerator failures. Quiescent settling zones near non-operational units developed and upon restart, bottom-scouring occurred often to the extent that the surrounding surface waters became visually black.

8.4 Heavy Metal Removals

The presence of heavy metals into and through the lagoon was monitored. Metals tested included iron, cadmium, cromium, copper, lead, zinc and mercury. Approximately 75 samples were evaluated for metals concentrations. Of these typically 65-70 samples per metallic constituent were used in the analysis. Lagoon influent and effluent heavy metals means and standard deviations are presented in Table 6. It appears that influent lead and zinc concentrations were reduced due to treatment in the lagoon.

Removal of heavy metals within the lagoon as indicated by the lesser average concentrations in the effluent would be expected to occur by settling. If so, continual increase in benthic sample concentrations should have occured. Benthic samples (just above the mud line) were routinely collected during the evaluation program at stations B1 and B3, which are located along the north side near the influent and effluent respectively. Ranges of benthic heavy metal results for stations B1 and B3 are presented in Table 7. The results are presented for three different periods of the evaluation program.

Substantial changes in lagoon flow rates have been shown to affect flow patterns developed in the basin. The effects of this phenomenon in combination with the increased lagoon loading rate at higher flows can be seen in the benthic samples metals concentrations. Early in the evaluation, inflow rates had averaged 3 mgd for a long period yielding long

TYPING GUIDE SHEET
BEGIN FIRST LINE OF TEXT HERE
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD: BEGIN SECTIONS HERE
6-1/2"
9-1/8"

TABLE 6

STATISTICS OF LAGOON HEAVY METALS CONCENTRATIONS

	Influent		Effluent	
	*	**	*	**
Fe	38.3	198.0	5.3	12.2
Cd	5.9	6.3	6.9	22.0
Cu	186.5	220	53.5	81.0
Pb	200.2	234	79.4	123.0
Zn	603.5	1763	120.3	261.0
Cr	107.7	191	90.6	160.0
Hg	2.52	2.18	2.45	5.0

* Mean (micrograms/l)

** Standard deviation

BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
3/8"

PAGE NUMBER
EPA-287 (Cin.)
(4-76)

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD; BEGIN SECTIONS HERE

TABLE 7

BENTHIC HEAVY METAL RESULTS

Analysis Period		Fe * B1	Fe * B3	Cr * B1	Cr * B3	Cd * B1	Cd * B3	Cu * B1	Cu * B3
A	High	270	172	3946	198	272	29	9620	630
	Low	50	22	107	95	16	20	155	48
B	H	154	150	4660	1640	353	343	20240	5350
	L	23	67	762	323	14	14	188	187
C	H	264	445	4550	7760	327	555	15000	22720
	L	89	47	840	520	70	35	2590	1430

		Zn * B1	Zn * B3	Pb * B1	Pb * B3	Hg * B1	Hg * B3
A	H	1288	188	8170	510	5.3	5.7
	L	40	145	9	34	-	-
B	H	21110	20730	8200	3660	9.0	.5
	L	9880	403	594	660	0.3	0.2
C	H	44200	76000	11090	16060	16.0	19.2
	L	13200	4720	3770	1300	2.2	0.8

Note:

A - January through March, 1979

B - April through June, 1979

C - July through September, 1979

*All concentrations are micrograms/dry solids

BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
6-1/2"
9-3/8"
3/8"

PAGE NUMBER
EPA-287 (Cin.)
(4-76)

detention times and optimum conditions for particulate settling. Examination of Table 7 shows that heavy metals concentrations during period A were substantially higher at station B1 (near the influent) than at station B3. It can be implied then, that most of the inorganic particulate settling occured at the influent end of the lagoon.

Increased flow into the lagoon has been shown, in other analysis, to change flow patterns directly affecting particular matter settling in two ways. Increased velocity head at the point of discharge has been shown to cause influent waters to flow to the far side of the lagoon before turning toward the effluent. This totally removes the direct flow from the area of station B1 such that this location no longer acts as the zone of initial settling. In addition the increase in turbulence appears to limit the extent of initial settling. Aerator operation has been shown to nearly completely mix the lagoon although some settling of particles is shown to occur by the reduction in TSS. The net result of increased flow and more extensive (periods B and C) aeration appear to be increase settling in locations along the lagoon south side and along the effluent end. Examination of Table 7 for periods B and C shows a general tendency for increased heavy metal concentrations at station B3 (near the effluent).

Correlation analyses were conducted on the following data sets: a) intra heavy metal concentrations-influent, b) intra heavy metal concentrations-effluent; c) influent heavy metal concentrations versus TS, TSS and BOD; d) effluent heavy metal concentrations versus effluent TS, TSS and BOD. The above data sets were also partitioned with respect to the three different periods of influent flow. Almost all of the correlation coefficients were insignificant at 95% confidence level. Zinc, lead and copper influent/effluent concentrations were highly inter-correlated. Data were too meager and incomplete to assess heavy metal removals during storm events when combined sewer overflow were pumped from the Magoun Avenue pumping station.

8.5 Stormwater Impacts

The first storm event monitored began at 6:17 A.M., on 6/29/79. Measured at the ECSD WWTP and ended at 8:05 A.M. cumulative rainfall equalled about 0.3 inches. Sampling of the lagoon influent began at 6:50 A.M. Initial flow rate to the lagoon was not recorded and it is unknown whether storm waters were being pumped from the Magoun Avenue Station at the time lagoon sampling had commenced. Estimated volume of stormwater pumpage from the Magoun Avenue Station to the lagoon is 0.7 MG.

The second storm event occured on 7/30/79. Cumulative rainfall for the storm event equalled 0.7 inches. Stormwater pumpage from the Magoun Avenue Station equalled 4.2 MG. The third storm evaluated occured on 8/23/79. Although the measured rainfall at the treatment plant was slight, visual observatins of rainfall in other sections of the drainage area were reported to have been more intense. Total flow pumped to the lagoon equalled 1.4 MG and appeared quite high for the 0.1 inch rainfall reported at the treatment plant.

Four days after the third monitored storm, another event began on 8/27/79 and rainfall equalled 0.3 inches in the drainage area. Initial sampling and flow measurements were performed prior to any pumping from the Magoun Avenue Station. These pumps then operated constantly for 1-1/2 hours and intermittently for an additional hour, delivering a total of 2.1 million gallons to the lagoon. Lagoon influent rates which were 15.6 mgd prior to the storm, averaged 34 mgd and peaked at 75 mgd during the storm. Peak effluent flows occured 1-1/2 hours after stormwater pumping began.

Recorded rainfall for each storm and total storm water volume monitored entering the lagoon are shown below.

Storm Event	Date	Recorded Rainfall Inches	Stormwater Volume Entering the Lagoon	Ratio MG:Inches
1	6/29/79	.3	.7	2:1*
2	7/30/79	.7	4.2	6:1
3	8/23/79	.1	1.4	14:1**
4	8/27/79	.3	2.1	7:1

* Higher volume of inflow known to exist

** Higher rainfall intensity reported in drainage areas outside measurement area.

Pumpage volumes to rainfall ratios determined for storms 2 and 4 agree quite well but differ for storm 1 and 3. Influent monitoring of the first storm began some time after actual inflow of storm water began. The measured volumes can be presumed to be less than the actual total storm inflow. The third storm displayed the reverse effect in which a low intensity storm provided a relatively high level of storm water inflow. It would be quite probable that an average rainfall of 0.2 inches could have occured given the thundershower nature of storms within this period yielding a similar volume to inches of rain ratio as storms 2 and 4.

Since the extent of these variations is only hypothetical no definite conclusions can be drawn. It appears, however, that storm water is pumped to the lagoon at a rate of .7 MG per tenth of an inch of rain.

The effects of the stormwater on lagoon operation were evaluated for immediate inter-lagoon effluent responses and for delayed responses on the subsequent days. Each of the storms exhibited similar responses within the lagoon. The extent of these responses increased with total stormwater flow. Details are presented for the fourth event for which data collection was the most complete.

During the pumping of stormwater into the lagoon on 8/27/79, the concentration of all pollutants in the CSO and treatment plant effluent varied widely from the prestorm influent values. Changes in influent and effluent flow rates and TSS and COD concentrations with time for this event are shown in Figure 18 along with long term 3 month averages for the lagoon influent and effluent. Influent concentrations of most constituents increase sharply during stormwater flow. Influent concentration increases above the long term averages indicate higher concentrations in the stormwater than in the treatment plant effluent. Increased effluent discharge reflects the additional storm loading, but changes in the pollutant concentrations within the effluent are minimal. At the end of frequent sample collection, influent concentrations had stabilized near prestorm levels, and effluent samples still remained relatively constant.

Under plug flow conditions, a substantial increase in effluent pollutant concentration would be seen for those constituents which had higher concentrations in the stormwater and/or those from the treatment plant which had been effectively removed in the lagoon and were now washed out without treatment. Any constituent of lower concentration in the stormwater than in treated lagoon effluent, could during storm conditions, result in effluent concentrations below typical levels. Inspection of effluent concentrations shown in Figure 18 and of samples taken 10 and 25 hours after initial outflow, showed neither of these effects, indicating that plug flow conditions were not occuring.

Assessment of lagoon treatment performance for wet weather runoff events is extremely difficult to accomplish for the following reasons. First, the pumped effluent from the wastewater treatment plant continuously varied such that the estimation of dry weather loadings would be subject to considerable error. Secondly, the automatic samplers monitoring both the influent and effluent were prone to mechanical malfunctions resulting in incomplete representations of storm events. Thirdly, the wet weather events were never totally monitored since the washout period was always substantially longer than the period during which the samplers were in operation. The hydrograph and the corresponding pollutograph for the storm events were never completely defined. Fourthly, the lagoon acts to retain pollutants during an overflow event. These retained pollutants are then treated in presumably a similar efficiency as occurs during dry weather. There may be

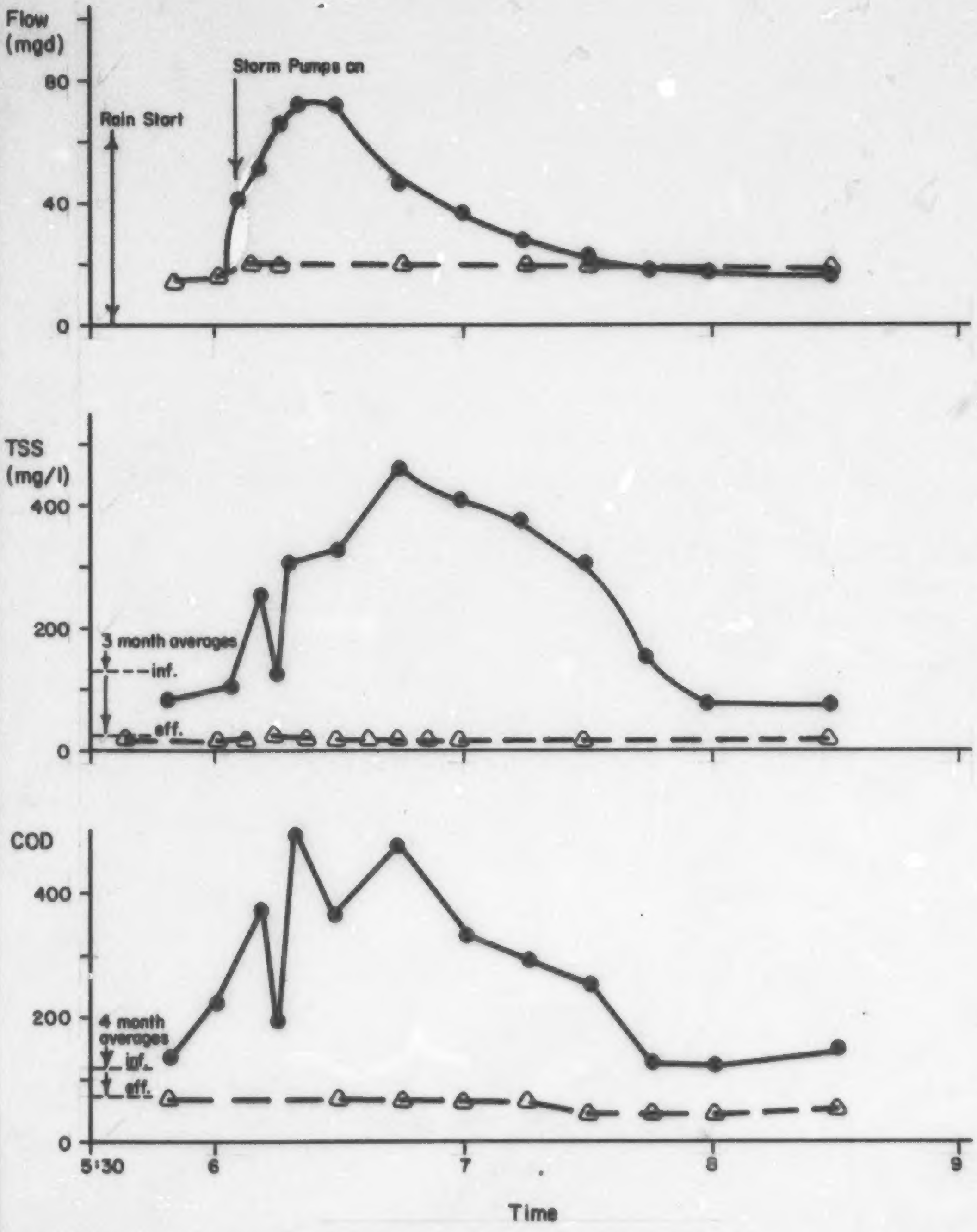


Figure 18. Lagoon effluent concentrations, storm of 8/27/79.

similar efficiency as occurs during dry weather. There may be some detrimental effects associated with contaminants in combined sewer overflows relative to contaminants in the dry weather polished secondary wastewater treatment plant flow but the monitoring program was not designed to determine these differences. Thus it is a difficult task to accurately estimate the overall performance of the East Chicago lagoon in treating pumped combined sewer overflows from the Magoun Avenue pumping station.

Table 8 summarizes the analysis of wet weather pollutant removal characteristics for the three storms occuring on 6/29/79, 7/30/79 and 8/27/79. The sampling results are incomplete for the storm occuring on 8/23/779 and are therefore not reported. Summaries of these three storms are contained in Table 8. Starting from the left, descriptions of the 14 columns in each of the three tables are as follows. Column 1 represents the measured influent mass during a storm event. This mass is a sum of the wastewater treatment plant effluent and the pumped stormwater from the Magoun Avenue pumping station. The second column represents the mass of dry weather wastewater treatment plant input for the period of the storm. This estimate is based on the typical dry weather wastewater treatment plant concentration occuring during the preceding week period to each storm event. Column 3 represents the pollutant mass input associated with the storm, that is, it is the difference between the total (col. 1) and the dry weather input (col. 2). Column 4 represents the average monitored lagoon effluent concentration during the storm event.

Column 5 is the measured lagoon effluent mass based on the average lagoon effluent concentration during the storm event and the average measured storm water effluent flow from the lagoon. This estimate reflects only loadings measured during the storm event and does not reflect the continuing washout effluent loadings not monitored by the sampling program. Column 6 is an estimate of the total lagoon effluent pollutant discharge over the time interval that the lagoon is affected by the storm event. This loading estimate is computed by scaling the measured lagoon effluent mass from column 5 by a multiplier defined by dividing the measured storm input volume by the measured storm output volume.

Column 7 is the average long-term dry weather lagoon effluent concentration. Column 8 is an estimate of the dry weather pollutant loading contribution discharging over the course of the storm event. This estimate is computed by using the average lagoon effluent concentration from column 7 times the estimated effluent volume discharging over the time required for the entire storm to pass. The volume output is estimated by the product of the time of measured volume output and the average lagoon effluent flow rate during the storm event.

TYPING GUIDE SHEET
BEGIN FIRST LINE OF TEXT HERE
DROPPED HEAD; BEGIN SECTIONS HERE
CENTER OF PAGE
TOP OF IMAGE AREA

TABLE 8

ANALYSES OF LAGOON CSO TREATMENT EFFECTIVENESS

STORM OF 6/29/79

	1 Total Wet Input (KG)	2 Dry Input (KG)	3 Storm Input (KG)	4 Measured Output During Storm (mg/l)	5 (KG)	6 Est. Total Output (KG)	7 Dry Output (mg/l)	8 (KG)	9 Wet Output (KG)	10 Storm Retained (KG)	11 (%)	12 Dry Rem. (%)	13 Treated Storm (KG)	14 Storm Treated (%)
COD	1352	672	680	45	160	2296	47	2281	15	665	98	43	286	42
TOC	635	325	310	34	120	1722	28	1377	345	-	-	32	-	-
TSS	1496	396	1100	46	163	2339	28	1363	976	124	11	85	105	9
VSS	753	103	650	19	69	990	19	918	72	578	89	75	433	67
TS	9922	7722	2206	1930	6884	98770	1660	80720	18050	-	-	11	-	-
NO_3	48	26	22	5.5	19	273	3.4	165	108	-	-	1	-	-
TP	10.2	4.1	6.1	.39	1.37	19.7	.25	12.2	7.5	-	-	77	-	-

BEGIN LAST LINE OF TEXT
3/8"
3/8"
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
PAGE NUMBER

EPA-287 (Cin.)
(4-76)

TYPING GUIDE SHEET
BEGIN FIRST LINE OF TEXT HERE
DROPPED HEAD, BEGIN SECTIONS HERE
CENTER OF PAGE
TOP OF IMAGE AREA

TABLE 8 (Continued)
ANALYSIS OF LAGOON CSO TREATMENT EFFECTIVENESS

STORM OF 7/30/79

	1 Total Wet (KG)	2 Dry (KG)	3 Storm (KG)	4 Eff. During Storm (mg/1)	5 (KG)	6 Est. Total Eff. (KG)	7 Dry Base (mg/1)	8 (KG)	9 (6-8) Storm Eff. (KG)	10 (3-9) Storm Retained (KG)	11 %	12 Dry % Rem.	13 Treated Storm (KG)	14 % Treated
COD	7755	1225	6500	26	477	1875	26	1458	417	6083	94	43	2616	40
TOC	2567	714	1853	28	518	2034	19	1093	941	911	49	32	291	16
TSS	7615	915	6700	36	668	2625	42	2358	267	6433	99	85	5469	82
VSS	3640	295	3345	17	306	1202	18	1010	192	3153	94	75	2364	71
TS	45700	27700	18000	1704	31460	123645	1696	95027	28619	-	-	11	-	-
NO3	150	56	94	4.3	79	310	4	224	86	8	8	1	.08	1
TP	48	7	41	.41	7.6	30	.45	25	5	36	88	77	28	67

BEGIN LAST LINE OF TEXT
3/8"
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
PAGE NUMBER
EPA-287 (Cin.)
(4-76)

TYPING GUIDE SHEET
BEGIN FIRST LINE OF TEXT HERE
DROPPED HEAD; BEGIN SECTIONS HERE
CENTER OF PAGE
TOP OF IMAGE AREA
BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA, OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
3/8"
PAGE NUMBER
EPA-287 (Cin.)
(4-76)

TABLE 8 (Continued)

ANALYSES OF LAGOON CSO TREATMENT EFFECTIVENESS
STORM OF 8/27/79

	1 Total Wet Input (KG)	2 Dry Input (KG)	3 Wet Input (KG)	4 Measured Output Eff. (mg/1)	5 (KG)	6 Est. Total Eff. (KG)	7 Dry Eff. (mg/1)	8 (KG)	9 Storm Eff. (6-8) (KG)	10 Storm Retained (3-9) (KG)	11 (%)	12 Dry Rem. (%)	13 Treated Storm (KG)	14 Wet Treated %
COD	4820	886	3934	59	1100	2310	55	1646	664	3270	83	43	1406	36
TOC	1288	238	1050	27.6	500	1050	24.5	739	311	739	70	32	236	22
TSS	3965	565	3400	18.7	340	714	20	599	115	3285	97	85	2792	82
VSS	1677	177	1500	7.77	140	294	6	180	114	1385	92	75	1039	69
TS	20300	12200	8100	1412	26000	54600	1516	42800	11800	-	-	11	-	-
NO_3	68	43	25	4.4	80	168	4.0	120	48	-	-	1	-	-
TP	27	5.8	2112	28	5.2	11	.26	7.8	3.2	18	85	77	14	65

TYPING GUIDE SHEET

TABLE 8 (Continued)

NOTES: ANALYSES OF LAGOON CSO TREATMENT EFFECTIVENESS

COLUMN	DESCRIPTION
1.	Total influent mass added during storm event (WWTP effluent + pumped storm volume).
2.	Dry weather WWTP input for period of storm (based on typical dry weather WWTP effluent concentrations).
3.	Estimated storm mass input Col.1-Col.2
4.	Mean measured lagoon effluent concentration during storm event.
5.	Measured lagoon effluent mass based on Col. 4 and average measured storm effluent flow.
6.	Estimated lagoon effluent pollutant discharge over time interval lagoon is affected by storm (Col.5 (storm volume in/ storm volume out)).
7.	Lagoon effluent dry weather average concentration.
8.	Estimated lagoon effluent dry weather pollutant load discharged during storm event , i.e., Col.7* estimated effluent volume discharging during time required for entire storm to pass.
9.	Mass of influent storm pollutant load in lagoon effluent i.e., Col. 6 - Col.8.
10.	Mass of influent storm pollutant load retained in lagoon, i.e., Col. 3 - Col. 9.
11.	Percentage of total storm mass retained. Col. 10 to Col. 3.
12.	Typical lagoon dry weather pollutant reduction performance (percentage)
13.	Estimated mass of retained storm pollutant load treated assuming retained mass is treated at same rate as dry weather loads.
14.	Percentage of total storm input mass load treated by lagoon.

EPA-287 (Cin.)
(4-76)

Column 9 represents the mass of stormwater load passing out the lagoon effluent, that is, it represents the difference between column 6 minus column 8. Column 6 is the total estimated effluent mass. Column 8 represents the total estimated dry weather mass discharging during the storm event. Column 10 represents the mass of the influent storm pollutant load retained in the lagoon, that is, it reflects the difference between column 3 which is the estimate of the storm mass input minus column 9 which is the estimate of the stormwater effluent loading. Column 11 represents the percentage of the total storm mass retained in the lagoon, that is, it is the ratio of column 10 to column 3. Column 12 represents the typical long-term lagoon dry weather pollutant efficiency. Column 13 represents an estimate of the mass of retained storm pollutant load treated assuming that the retained mass is treated at the same efficiency as dry weather loads. And finally, the percentage of the total storm input mass load treated by the lagoon, that is including both retention and dry weather treatment is represented in column 14.

It can be seen from the inspection of Table 8 for the 3 reported storms that the lagoon provides a reasonable degree of treatment for several contaminants for the three wet weather events. The removal results from Table 8 are summarized in Table 9. The overall percentage removal per event together with a range of treatment removal are given in Table 8. The range of removal for COD is 36-42%. The range of TOC is 16-22%. TSS removals are erratic ranging from 9 to 82%. The range of pollutant removals for VSS and TP are 65-71%.

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD, BEGIN SECTIONS HERE

TABLE 9

SUMMARY OF LAGOON WET WEATHER TREATMENT PERFORMANCE

Pollutant	% Overall Removal			Range
	Event A	B	C	
COD	42	40	36	36-42
TOC	-	16	22	16-22
TSS	9	82	82	9-82
VSS	67	71	69	67-71
TP	-	67	65	65-67

A - storm 6/29/79

B - storm 7/30/79

C - storm 8/29/79

6-1/2"
9-1/8"
BEGIN LAST LINE OF TEXT
3/8"
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS

PAGE NUMBER
EPA-287 (Cin.)
(4-76)

CHAPTER 9

DISINFECTION PROGRAM RESULTS

9.1 Foreword

Description of the chlorine/chlorine dioxide disinfection system used in East Chicago were presented in Chapter 4.0. Descriptions of the disinfection system calibration activities were presented in Chapter 6. For purposes of presentation the disinfection evaluation program is subdivided into three separate phases. The first phase spanned the period of May through July, 1979 and dealt with routine bacterial monitoring of lagoon disinfection system for dosage rates established and maintained by ECSD to meet WWTP NPDES permit requirements. The data collected during this period has limited practical value due to continuous malfunction of the disinfection system. Overview results are presented in section 9.2. The second phase of work conducted in August and September, 1979 consisted of a controlled set of varied chlorine/chlorine dioxide dosage experiments. This program was aimed at developing various equivalent dosage combinations for comparable fecal coliform kill effectiveness. Summary results of these experiments are presented in section 9.3. The final phase of work dealt with a special bench scale study using chlorine/chlorine dioxide aimed at augmenting data deficiences of the second phase effort. These results are presented in section 9.4.

9.2 Routine Disinfection Evaluation Program (May - August 1979)

Installation of East Chicago's chlorine/chlorine dioxide disinfection system and initial recommendations for dosages of the sequentially applied disinfectants were based on earlier bench scale work treating combined sewer overflows. (1) Although the East Chicago lagoon is designed to handle storm overflows. Much of the water treated is secondary WWTP effluent. Disinfection facilities are located to provide treatment after stabilization in the lagoon of stormwater and wastewater. Minimal amounts of stormwater bypass the lagoon and are disinfected without prior treatment. Dosages based on optimum requirements for untreated combined sewer overflows were established by ECSD as the nominal initial dosage setting that would be required during the adjusted evaluation program.

The disinfection system was designed to provide nominal chlorine dosage of 8 mg/l followed by 2 mg/l of chlorine dioxide within a two minute contract period. The ECSD WWTP effluent standards are based on weekly and monthly maximum average total coliform values of 200 and 400 colonies per 100 ml, respectively. Estimation of lagoon disinfection dosage rates were difficult since the lagoon dry weather pumps handle only a portion of the WWTP effluent with bypassed high dry weather flow mixed back with disinfected lagoon effluent in earthen channel leading to the Grand Calumet river. Excess disinfectant residual in lagoon effluent was at times necessary to ensure that mixed channel flow met bacterial standards. Dosage rate estimation was further confounded by the variability characteristics of the WWTP effluent discharge. Variation of WWTP plant flow up to 4 mgd occurred over 15 to 20 minute intervals. It was often found that the lagoon inflow pumpage rate was exceeded during the high flow portion of these cycles, but not during minimum flow period. Samples collected by treatment plant personnel during low plant flow periods were almost entirely disinfected lagoon effluent. Bacterial analyses of these samples indicated low colony counts. Samples collected during high treatment plant flow periods, with significant lagoon bypass, represented partially mixed effluent with occasional high coliform counts. Lagoon effluent disinfectant dosages were set to provide sufficient residuals for treatment of the bypassed treatment plant flow. Disinfectant residual beyond specified limits was provided during low plant flow and no lagoon bypass. Although this problem would be resolved after return to normal operation (the pumping capacity of the lift station would handle most plant flows), occasional occurrences of increased plant flow would cause similar effects especially during light rain periods. Persistent technical problems precluded long-term evaluation of the chlorine/chlorine dioxide system disinfection effectiveness.

Components of the disinfection system were delivered, installed and calibrated by the equipment manufacturer and ECSD throughout the spring of 1979. The system went on-line in May, 1979 but did not properly function throughout the remainder of the evaluation program. During the first several months of operation the diffusers clogged within hours. The automatic controller meant to establish disinfection dosages as a function of lagoon discharge did not work despite continuous repair by manufacturer. As a consequence, actual dosage rates were not known for any period of time. Lagoon stage/discharge calibration curves earlier developed by ECSD were used to establish manual dosage settings. Careful examination of these curves near the end of the evaluation program revealed errors in computing discharge from stage readings.

Bacterial analyses of the lagoon effluent was monitored every other day during the evaluation period. Three bacterial types were monitored including total coliform, fecal coliform and fecal streptococci. A statistical evaluation of the colony counts for each bacteria type and relationships among types is shown in Table 10. Residual bacteria counts following secondary and lagoon

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER
TOP OF IMAGE AREA
DROPPED HEAD:

TABLE 10

STATISTICAL ANALYSIS LAGOON BACTERIAL LEVELS

A. Lagoon Effluent (colonies/100 ml)

Type	N	Max. No.	Min. No.	Arith. Mean	Std. Devn.	Coef. Var.(%)	Geometric Mean
TC	104	$2.8x10^6$	500	$1.72x10^5$	$3.9x10^5$	228	$5.66x10^4$
FC	104	$2.8x10^4$	35	$3.3\ x10^3$	$4.7x10^3$	141	1720
FS	105	$7.2x10^4$	11	$5.1\ x10^3$	$8.7x10^3$	171	2390

B. Linear Correlations Between Lagoon Effluent Bacteria Types

	N	Linear Correlation	t
TC VS FC	104	.09	.92
TC VS FS	104	.06	.61
FC VS FS	105	.09	.92

C. Linear Correlations Between Bacterial Percent Removals

Untransformed	N	Inter-Correlation	t
TC VS FC	116	.33	3.9
TC VS FS	115	.28	3.2
FC VS FS	116	.31	3.7
Log Transformed			
TC VS FC	116	.58	9.3
TC VS FS	115	.51	7.3
FC VS FS	116	.63	11.1

TC - Total Coliform
FC - Fecal Coliform
FS - Fecal Streptococci
t - Students "t" statistic

BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
3/8"

EPA-287 (Cin.)
(4-76)

treatment were roughly one percent of typical raw sewage values, as indicated by the arithmetic and geometric means. As shown in Table 10, the linear correlations between bacteria types were poor. Analysis of the t-statistic implied that there was no correlation between the three types.

Post chlorination colony counts were generally low since most of the time sufficient disinfectant was applied to the lagoon effluent to ensure adequate disinfection of the entire plant effluent. Often, effluent was overdosed to ensure a sufficient disinfectant residual to treat the bypassed plant flow. Generally this overdose condition led to near complete disinfection of the lagoon effluent. Comparative analysis of post-disinfection colony counts were not tabluated for this reason. There were a sufficient number of occasions in which equipment malfunctions occurred and limited dosage resulted such that the bacteria removal over the evaluation period averaged 90 percent. Linear correlations for untransformed (arithmetic) and logarithmic percent bacterial removals are also presented in Table 10. The t-statistics for the log transformed data indicate reasonable correlations at the 95% confidence level.

9.3 Chlorine/Chlorine Dioxide Kill Studies

During the evaluation period, various combinations of chlorine and chlorine dioxide disinfection were used to determine optimum dosages for reducing lagoon effluent fecal coliform levels to 1 colony/ml. Descriptions of the chlorine system and the chlorine dioxide generator system were presented in Chapter 4. Calibration efforts to determine actual dose and production levels were presented in Chapter 6.

Initial dosage settings were based on previous pilot scale disinfection of combined sewer overflows. Optimum dosages recommended from that study were 8 mg/l Cl_2 followed by 2 mg/l ClO_2. (1) Results of early disinfection analysis in this study showed that adequate coliform reductions could be obtained with dosage of 3 to 4 mg/l Cl_2 acting within a 3-minute detention time.

Experiments in this program began using only chlorine. Dosages of 2.5 - 6.5 mg/l Cl_2 were added to yield background data for determination of actual ClO_2 efficiency during detailed evaluation of the flow monitoring system. It was discovered that the actual lagoon flow rates were higher than the intended levels during the disinfection experiments. On this basis, chlorine dosages were re-determined to be only two-thirds those originally intended.

Effluent coliform levels varied widely throughout the study period with fecal coliform colony counts ranging from 600 to 200,000 per 100 ml. This variance created difficulty in the overall evaluation since equivalent kills as determined on a percentage or log removal basis would actually yield a wide range of effluent levels.

Reduction in fecal coliform levels at several chlorine dosages with no sequential ClO_2 addition are shown in Figure 20A. Although the range of influent coliform levels preclude the direct comparison of treated effluent counts the slopes of lines do provide reduction rate information. Inspection of Figure 19-A shows that the rate of reduction increases with increasing chlorine dose. Average Chick's Law rate constants were determined to be .014 at a chlorine dose of .8 mg/l, .03 at 1.3 mg/l Cl_2 and

The effects of sequential ClO_2 addition are defined in Figures 19-B,C and D. In each of the three plots, the chlorine dosage is nearly constant and ClO_2 dosages cover a wide range. Delay times between sequential doses varied somewhat due to variations in lagoon flow rates, and fixed Cl_2 and ClO_2 diffuser locations. The plots are based on the total reaction time after chlorine addition with the times of chlorine dioxide addition indicated by arrows.

Direct comparison of residual colony counts after disinfection is not useful due to wide variations in lagoon effluent coliform values. It appears that there are no distinctive trends seen in any of the plots based on the slopes of the curves indicating little difference in disinfection efficiency with sequential chlorine dioxide addition.

9.4 Bench Scale Chlorine/Chlorine Dioxide Experiments

The previous bench scale study (1) indicated optimum sequential dose delays of 15 to 30 seconds. The full scale disinfection operation at East Chicago could not be analyzed under these conditions due to the fixed locations of the diffusers and limited lagoon effluent flow. It became clear, after much data analysis, that little advantage was seen with sequential ClO_2 addition after delays of 45 to 90 seconds. A small-scale bench study was performed in this study allowing the desired sequential delay times of 15 to 30 seconds.

As background data chlorine was added in dosages of 1, 2, 3, and 4 mg/l with no sequential ClO_2 addition. Next, sequential additions of ClO_2 in dosages of 0.5 mg/l to 3.0 mg/l were added after 30 seconds delay to samples pre-chlorinated with 1 mg/l and 2 mg/l Cl_2. Selected sequential dosage combinations were also applied using a 15-second delay period. The results of the bench study analysis are illustrated in Figure 20-A. As with the full-scale operation, the chlorine performance line indicates increased disinfection with increased chlorine dosage. The remaining lines are isobars of sequentially added ClO_2 dosages. The bench study data does indicate increased disinfection with

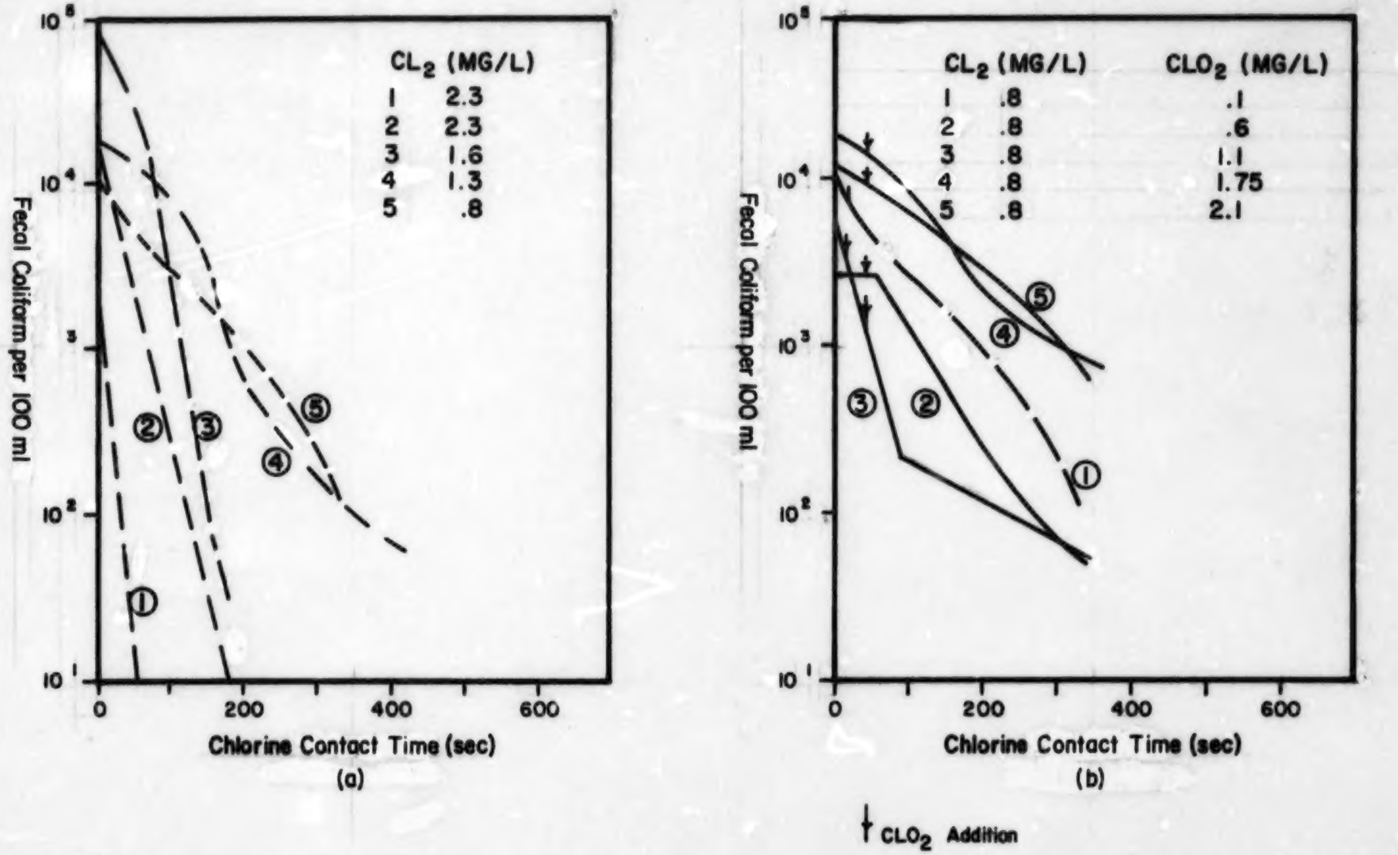
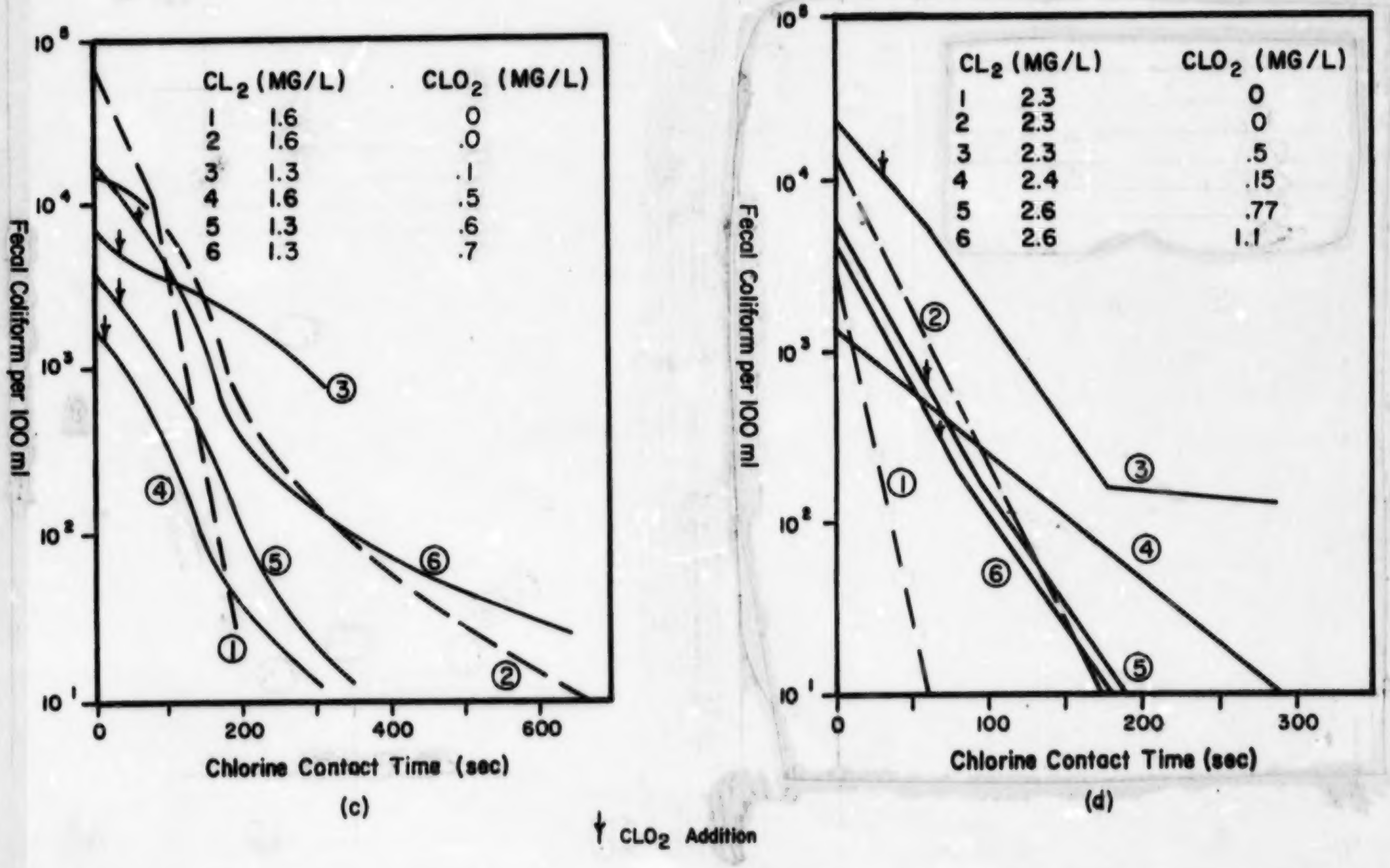


Figure 19. Sequential chlorine/chlorine dioxide disinfection results (full scale)

Fig 3 & 4



(c)

(d)

Figure 19. Sequential chlorine / chlorine dioxide disinfection results (full scale) - continued.

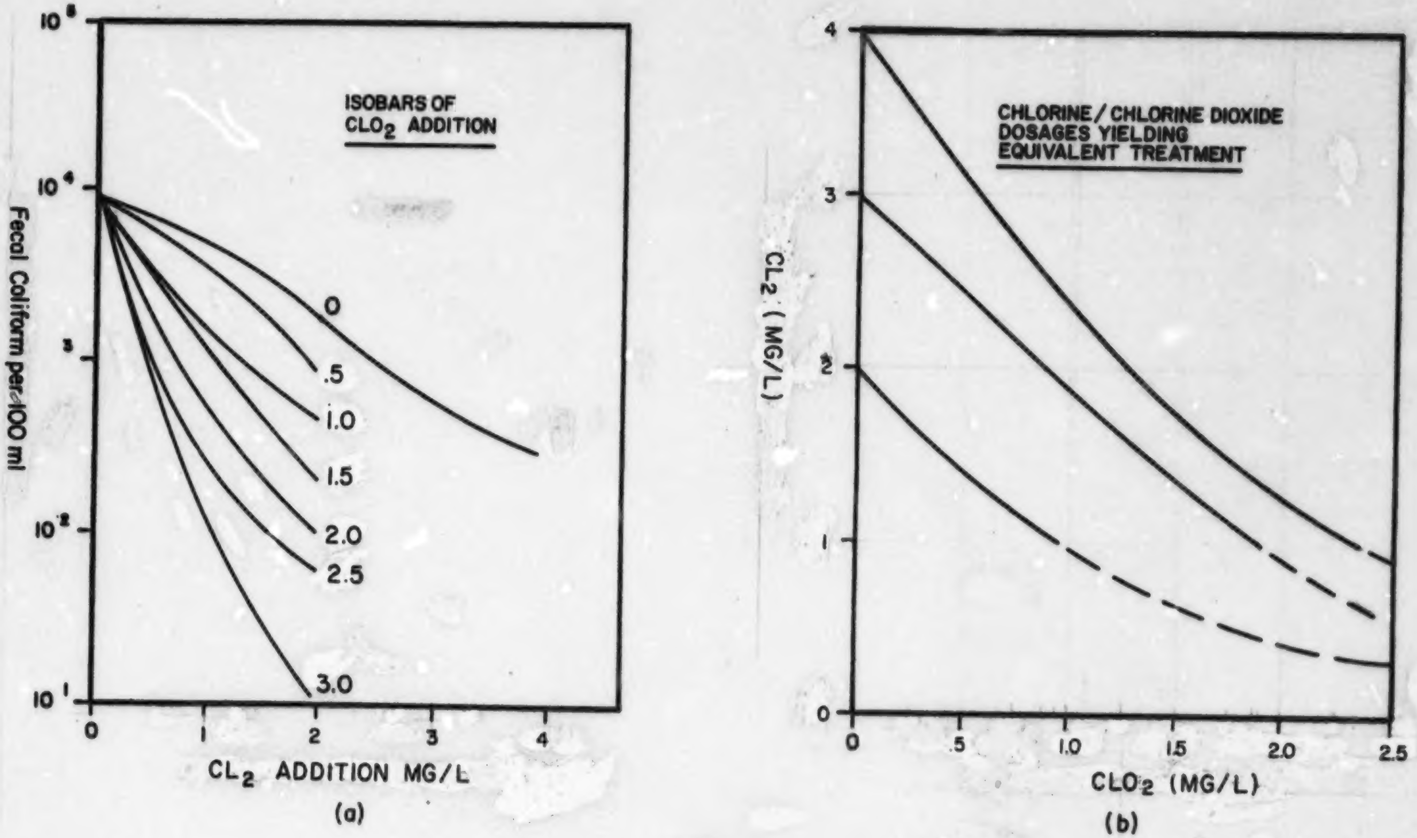


Figure 20. Sequential chlorine/chlorine dioxide disinfection results (bench scale operation).

sequential ClO_2 addition and the benefit of limiting the delay period between doses from 30 to 15 seconds. From this plot, equivalent dose combinations were determined and are presented in Figure 20-B. This study disinfection system utilized ClO_2 addition 30 seconds after initial Cl_2 addition and had a total reaction time of three minutes. The curves shown in Figure 20-B indicate dosage combinations which will yield equivalent fecal coliform removals. For any desired residual fecal coliform level after disinfection, the required dose of chlorine could be determined from Figure 20-A. The optimum combination of Cl_2/ClO_2 could be selected from the appropriate equivalence line in Figure 20-B considering pertinent localized cost information.

CHAPTER 10

POTPOURRI OF MISCELLANEOUS TOPICS

10.1 Foreword

A number of miscellaneous observations, analyses and experiments are presented in this chapter. Estimates of lagoon solids deposition rates based on measured total solids removals are presented in section 10.2. Visual observations noting wet weather related (CSO) solids accumulations in the influent channel from the Magoun Ave. pumping station are given in section 10.3. Comments on the general absence of algae production noted during the evaluation program are given in section 10.4. Experimental observations using the respirometer technique to evaluate BOD are presented in section 10.5. Results of oxygen uptake rate measurements using the respirometer technique are given in section 10.6.

10.2 Rate of Lagoon Solids Deposition

Limited total solids removals led to an empirical evaluation of sediment buildup rates within the lagoon. Average influent total solids concentration and average removal rates were reported in section 7.2. These data were used to determine the total mass of solids deposited in the lagoon during each of the three flow evaluation periods. The total solids reductions for the three flow evaluation periods were 220, 316 and 55 mg/l for the 3.0, 11.3 and 18.8 mgd flow rates respectively. Total mass of deposited solids equalled 1.26, 0.93 and 0.63 million pounds of materials for each period. Using a gross density of 70 pounds/cubic foot and assuming that the material was distributed uniformly over the lagoon resulted in depths of accumulation of 0.34, 0.25 and 0.17 inches per period, totalling to 0.76 inches.

10.3 Solids Deposition in the CSO Influent Channel Clean Out Pit

The influent channel to the lagoon permits stormwater flow to pass over a concrete pit before merging with the pumped lagoon inflow from the secondary plant. Deposited grit and organic material from this catchment device has not been removed after each storm. The pit is approximately 60 feet by 31 feet and 3 1/2 feet deep and contains no drainage facility. Solids accumulations had filled roughly half the pit volume by the end of the evaluation periods while the remaining volume contained residual storm water. The presence of algae growth in the overlying water in the pit indicated high nitrogen and phosphorus levels.

The functional significance of the wet weather solids trap is two fold. It is clear that the pit removes large quantities of solids which would otherwise be deposited in the lagoon. This feature deems its presence as a valuable asset to the overall operation and warrants that means be taken to optimize its effects. Functional optimization is the second area of concern. Removal of residual solids after each storm event would increase the capacity for solids collection during each event. Residual solids within the pit at the onset of a storm event can be expected to be intermixed with stormwater flow by scouring and deposited in the lagoon. Solids not scoured and intermixed will simply limit the effective area of the pit.

10.4 Algal Production and Inhibition

In a typical high rate aerobic pond, algae photosynthesis is utilized for a large part of the organic waste reduction. At the East Chicago installation nitrate removal was minimal and all phosphorus removal appeared to result from particulate settling. Occasional algal population counts were made in the lagoon with varying results. During the latter part of May, 1978, algal counts indicated populations over 1 million per 100 ml. In mid September algal colonies numbered less than 2000 per 100 ml.

The presence of toxic substances in the lagoon influent was considered as a possible source of this variation. Copper, which was monitored in the lagoon, is commonly used as an algaecide in the form of copper sulfate. This compound has been reported to be toxic to most algal organisms in concentrations of 0.1 to 0.5 ppm. Copper concentrations in the lagoon influent averaged .177 ppm over the entire evaluation period during which actual levels varied widely as reflected by the computed coefficient of variation of 1.18. These results indicated that toxic levels of copper were present in the lagoon. Copper removal rates were high within the lagoon and averaged about 70 percent over the entire study period. It was noted during benthic sample analysis that high concentrations of heavy metals were accumulating with the inference that the remocal mechanism was particulate settling.

A review of lagoon operations during the algal population analyses indicated little difference in influent copper concentrations. The most pronounced difference found was aerator operation. The presence of high algal populations was observed at the end of a long period without aeration. Low algae counts were determined under partial aeration conditions. On the basis of algae production and toxicity it appeared that mixing conditions induced by aeration creates the combined adverse effects of limiting colony residence time in sunlight areas by continual surface water changeover and promoted contact of algae with toxins such as copper which could otherwise have settled out.

10.5 Evaluation of BOD by Respirometer Technique

Although BOD is one of the standard organic pollutional indicators, the inherent time required to accomplish the test in addition to other limitations have spurred attempts to substitute other means of evaluation. In this study COD, TOC, BOD_5 and BOD as measured by a respirometer were monitored. Poor correlations between each of these parameters resulted during the subsequent evaluation of data. In the case of COD it is well documented that this parameter includes oxygen demands of organic compounds which are not readily biodegradable. Concentrations variations of these substances led to the poor correlation between the BOD and COD values. Analysis of total organic carbon requires very small sample sizes. It seems possible that with the high concentrations of dissolved solids these small aliquots may not have been representative of the original sample in addition to detecting substances other than those determined by the BOD test.

Instrumentation used was an Oceanography Internation Corporation E/BOD Respirometer System. The system removes any CO_2 produced by absorption with KOH pellets creating a vacuum and triggering the electrolyte production of oxygen. Cumulative oxygen production is measured and recorded by a remote printer at chosen time intervals.

Evaluation of oxygen demand by the respirometer technique showed little improvement over the use of COD and TOC. The results of several BODe evaluations performed on influent/effluent samples are illustrated in Figure 21. Each curve is typical of many cumulative oxygen demand tests performed. Some samples were noted to display immediate oxygen demands whereas others demonstrated delay periods of up to several days. Samples then proceeded to exhibit oxygen demands which were generally, at a steady rate. Most samples eventually reached a plateau in the cumulative demand plot signifying the end of oxygen demand. On many occasions samples which appeared to plateau one or several times, continue to exhibit continued demand with time.

The difficulty in relating BODe to BOD_5 results or comparing BODe results for different samples lies in the inability to define a certain point on the plot as the final value. For those cases where demands cease for long periods, their final cumulative value can be justified as the appropriate choice. For others, plots exhibitions, no plateau or numerous

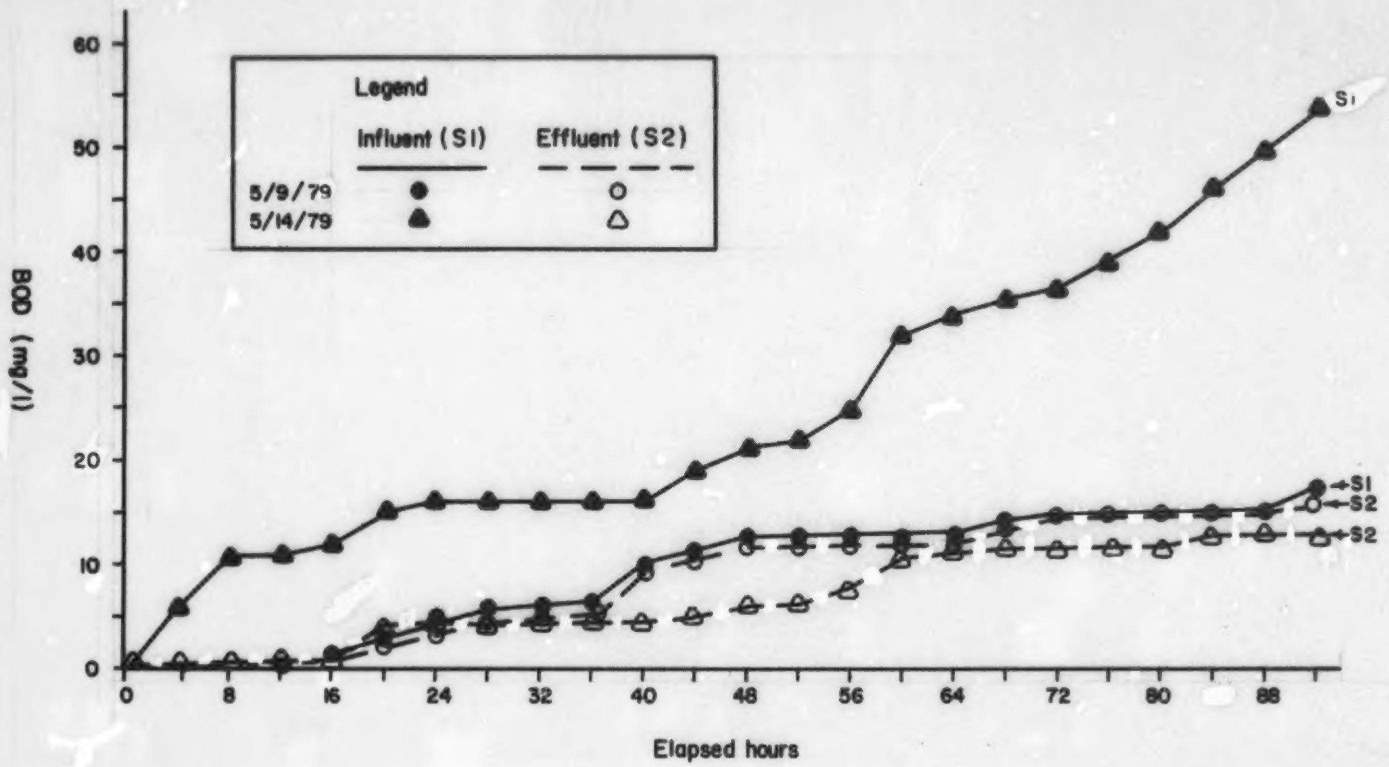


Figure 21. Typical lagoon influent/effluent BOD determinations.

plateaus renders this choice impossible. In many cases oxygen demand continued for periods exceeding four days such that this evaluation demonstrated no time advantage over the 5 day test. In this study plots rendering no plateau were evaluated at the 60 hour interval. This value was chosen as the recorded demand rate, since most samples which stabilized, did so by this point.

10.6 Oxygen Uptake Rates

The rate of oxygen uptake using the resprometer described in the previous section was determined during the evaluation period to generally depict oxygen update at various locations within the lagoon. This information is useful in understanding the dynamics of oxygen depletion in the lagoon. Throughout the evaluation period, influent and effluent samples were tested every other day. Near the end of the evaluation period, the rates of oxygen uptake were determined throughout the lagoon. Samples were collected at each station and tested with the resprometer. Oxygen uptake rates were determined for a period of 24 hours which was observed to portray representative start up, lag and initial demand rate characteristics. The variability of influent and effluent oxygen uptake is depicted in Figure 22. The results for samples collected on September 12, 14, 16 and 18, 1979 indicate high oxygen demand rates in both the influent and effluent samples. By 9/14/79, influent pollution constituents had lessened yielding a lower demand rate. Lag time within the lagoon resulted in a delay or a displacement of the 9/12/79 high oxygen demand to an extent that the effluent on 9/14/79 actually had a higher demand than the influent sample. On 9/16 and 9/18, influent flows again contained low demand rates. By this time, the combined effects of lagoon treatment and displacement resulted in low effluent demand levels. The observed trend clearly demonstrates that any overall removal analysis must be performed by averaging concentrations over a sufficiently long time period.

During this period, all regular monitoring stations were sampled and oxygen uptake experiments conducted for 4 depths, including the surface, the 10 foot, the 25 foot and the benthic level. Low or zero oxygen demands were found in the upper 3 samples at stations 1 and 2. In order to ensure bacterial acclimation to the waste, tests on stations 3 and 4, included seeding of the samples with 1 ml of lagoon influent. Rates of oxygen uptake at several depths at samples taken at station 4 are shown in Figure 23 down to 25 foot level exhibited similar demand rates and initial lag times. The oxygen uptake curve for the benthic sample was considerably higher, equaling about 106 mg/l per day. Oxygen demands for benthic samples collected at four locations are shown in Figure 24. Initial lag times varied somewhat yielding varying demand rates for the first few hours. The slopes of the line indicate the oxygen uptake rates. Stations 1 and 2 exhibit similar demands after the initial lag period.

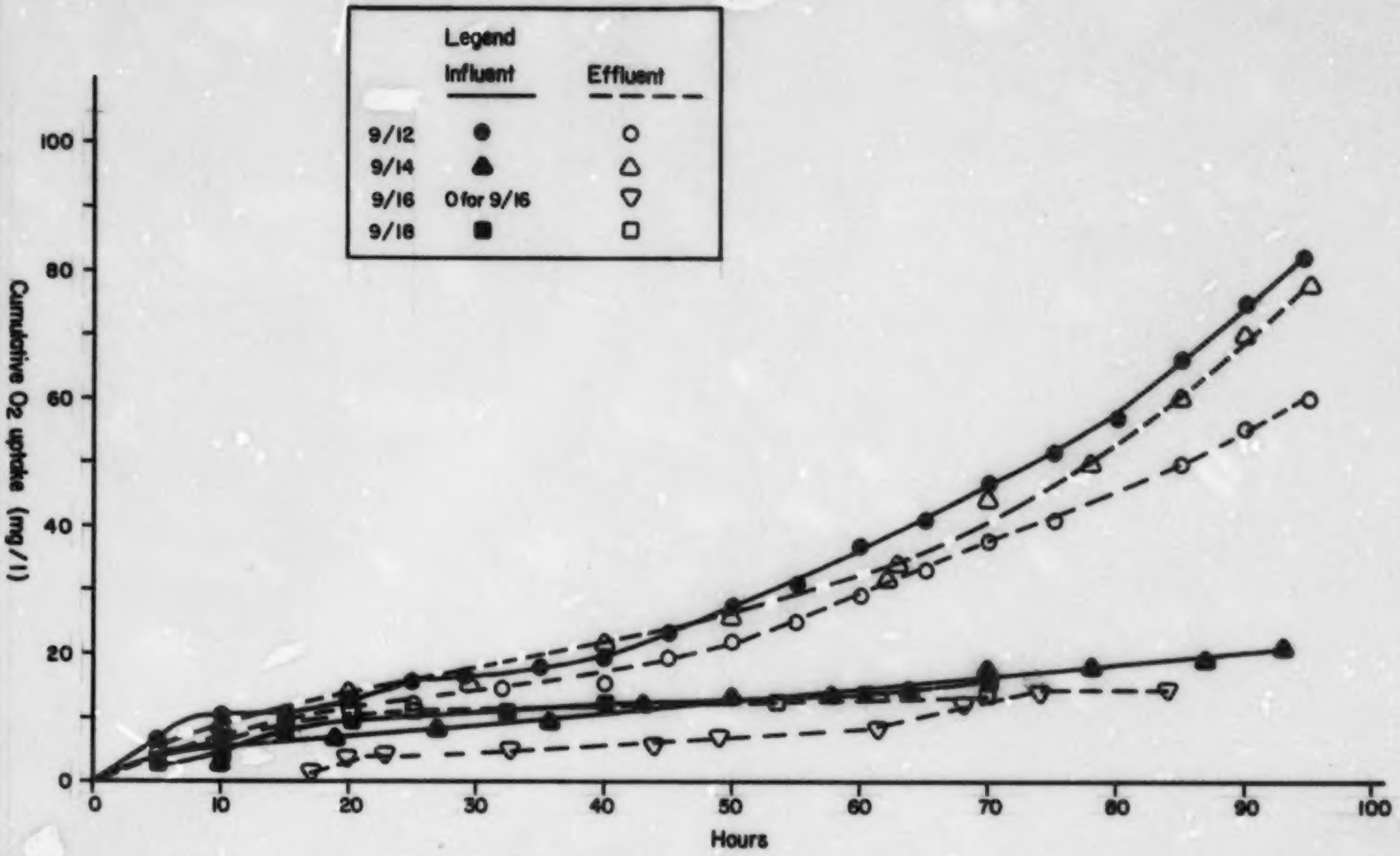


Figure 22. Lagoon influent/effluent oxygen uptake versus time.

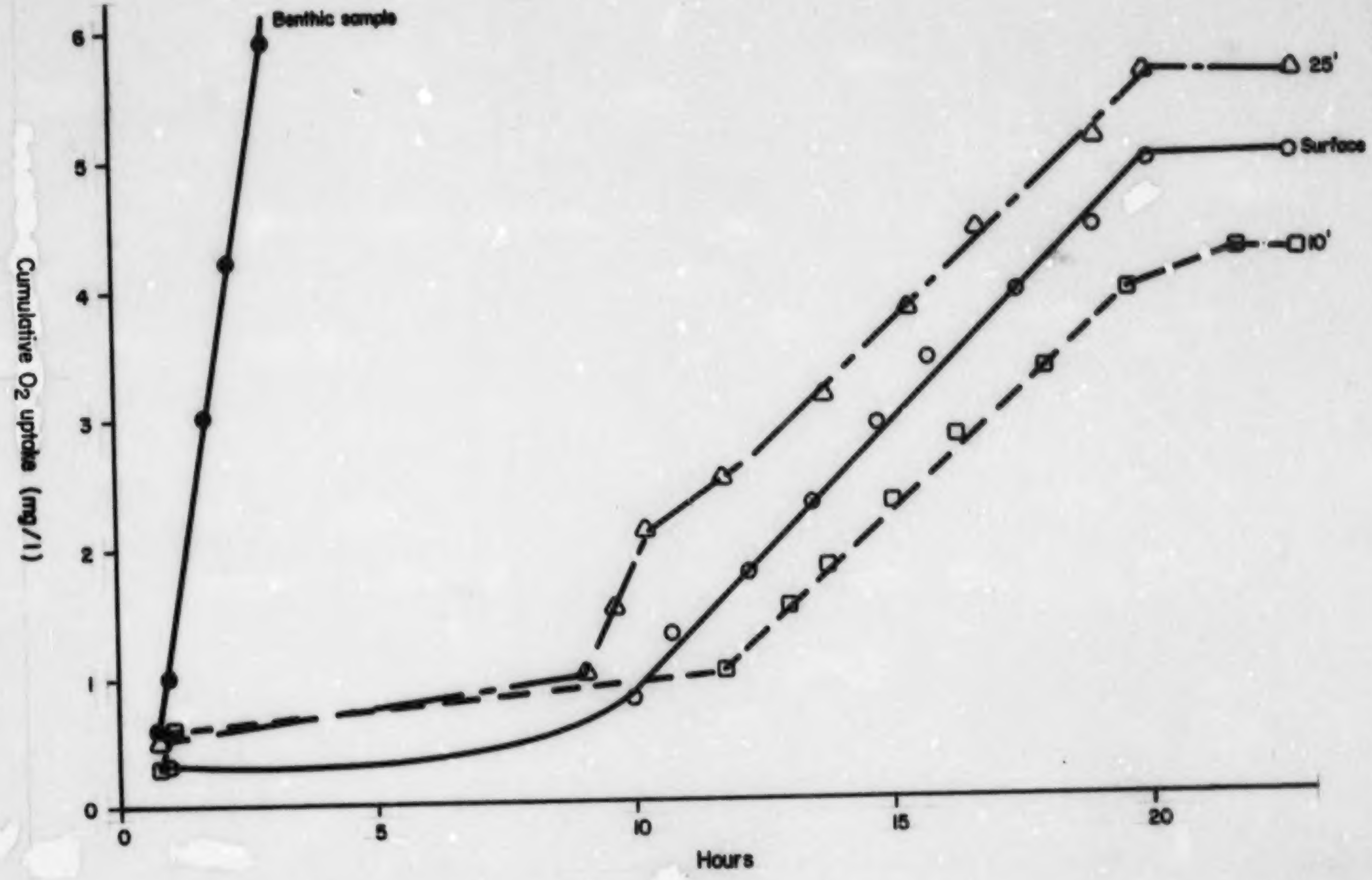


Figure 23. Oxygen uptake at station 4 versus time.

Oxygen uptake rate shown in Figure 24 for station 3 is much higher than the rate shown for station 4, suggesting that the degree of sedimentation is greater near station 3 as lagoon waters flow by this point lying directly between the lagoon influent and effluent. In addition, a review of the aerator performance in the station 5 area indicated long periods of down time suggesting a higher probability of sedimentation than in area 4.

The oxygen uptake experiments show characteristic oxygen demand increases and decreases through the lagoon following oxygen demand fluctuations in the influent. These fluctuations can be followed along the lagoon from the influent to the effluent end. Oxygen demands are relatively equivalent from the surface to the 25 foot depths at any sample station. Benthic sample demands are a function of the prevailing degree of depositing pollutants and are comparable at stations 1, 2, and low at station 4, which is offset from the main flow of lagoon waters under most circumstances.

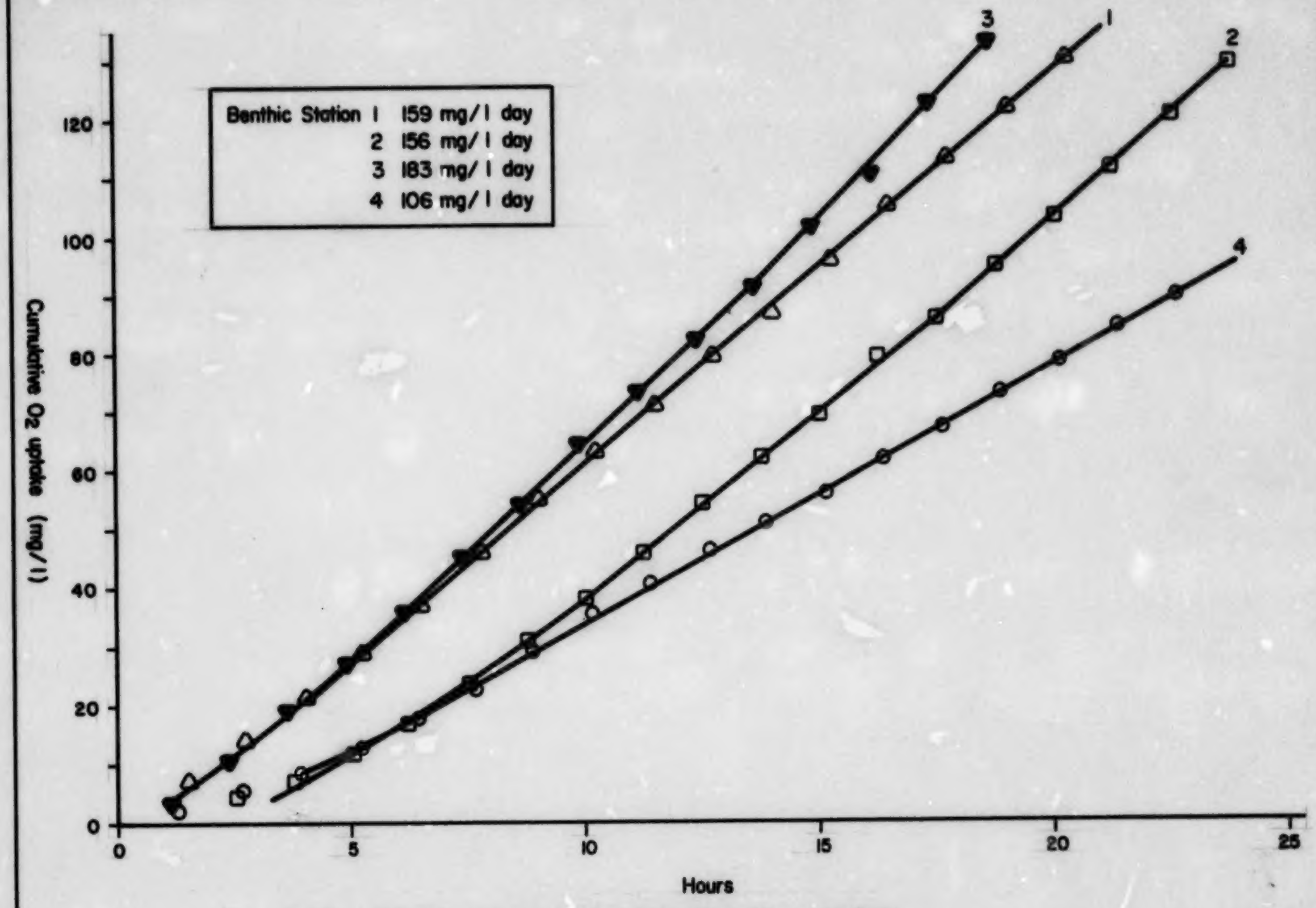


Figure 24. Benthic oxygen uptake curves at various locations.

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD; BEGIN SECTIONS HERE

REFERENCES

1. O'Brien & Gere Engineers, Inc. Disenfection/Treatment of Combined Sewer Overflows Syracuse, New York. U.S. EPA Grant No. S802400, Department of Public Works, Division of Drainage and Sanitation, County of Onondago, New York. 1979.

2. Instruction Manual, Dissolved Oxygen Meters; YSI Instruments, Yellow Springs, Ohio.

3. Standard Methods for the Examination of Water and Wastewater, 14th Edition; APHA-AWWA-WPCF, 1975.

4. Equipment Manual, Respirometer, Oceanographic International College Station, Texas.

5. Equipment Manual, Automatic Ampule Method, Oceanographic International; College Station, Texas.

6 1/2"
BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA; OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
3/8"

PAGE NUMBER
EPA-287 (Cin.)
(4-76)